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PROCEEDINGS

SEEPAGE SYMPOSIUM

Phoenix, Arizona

February 19 - 21, 1963

Agricultural Research Service

UNITED STATES DEPARTMENT OF AGRICULTURE

FOREWORD

Despite the generally accepted importance of seepage as a major problem in the conservation and management of irrigation water supplies, very few technical conferences have ever been held on the subject. During the year 1962, a number of people from a number of organizations commented on the desirability of holding such a conference.

Because of these comments and the Division's interest and responsibilities in this field, the Division decided to organize and sponsor a symposium on seepage under the leadership of its U.S. Water Conservation Laboratory, to be held at Phoenix, Ariz. The response far exceeded our expectations and conclusively demonstrated widespread interest in seepage phenomena and seepage problems. This interest was shown to be far broader than the field of irrigation alone. Many subjects were proposed for discussion at a future symposium.

The success of this Seepage Symposium was due to the excellent cooperation received from everyone who participated. Special thanks are due to Lloyd E. Myers, director, U.S. Water Conservation Laboratory, to Chairmen Clifford A. Pugh, Sol D. Resnick, and Dell G. Shockley, who kept the sessions running smoothly, and to the authors of the many excellent papers presented. Commendation is due Robert J. Reginato, who planned and managed all physical arrangements.

Director
Soil and Water Conservation Research Division

January 1965

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SESSION 1.--CHAIRMAN, CLIFFORD A. PUGH

FARM SEEPAGE PROBLEMS IN PONDS AND SMALL IRRIGATION RESERVOIRS[1]

J. J. Coyle[2]

Loss of water from farm ponds and small irrigation reservoirs by seepage is a serious problem facing Soil Conservation Service technicians in their work with farms and ranchers through the nearly 3,000 Soil Conservation Districts.

Small water-impounding structures offer the only feasible source of water on many thousands of farms and ranches. These structures must supply the major portion of the water needed for livestock, irrigation, orchard and crop spraying, fire protection, recreation, and many other uses. In some areas they provide the only source of water for domestic use.

Fortunately, it usually is possible in most areas to find a location on a farm or ranch where site conditions are such that water can be impounded with very little loss by seepage. This is true generally of the more than 1,200,000 ponds and small irrigation reservoirs on which the Soil Conservation Service has given technical assistance.

There are areas, however, where it is very difficult to find suitable sites. The boundaries of these problem areas correspond fairly well with the boundaries of several of the Major Land Resource Areas used by the Soil Conservation Service to localize problems in soil and water conservation. This makes it convenient to relate the following discussion of problems to Major Land Resource Areas as shown on the map in figure 1. A land resource area may be defined as a geographic area of land, usually several thousand acres in extent, characterized by a particular combination or pattern of soils (including slope and erosion), climate, water resources, land use, and types of farming.

The following discussion has also been arranged by groups of states as served by the six Engineering and Watershed Planning Unit offices of the Soil Conservation Service as shown in figure 1. Most of the information in this paper was contributed by those offices.

Engineers and technicians of the Soil Conservation Service examined the sites and supervised construction of about 56,000 ponds and small irrigation reservoirs in fiscal year 1962. A great majority of these were cost shared under the Agricultural Conservation Program. This brings to about 1,200,000 the number of such structures on which

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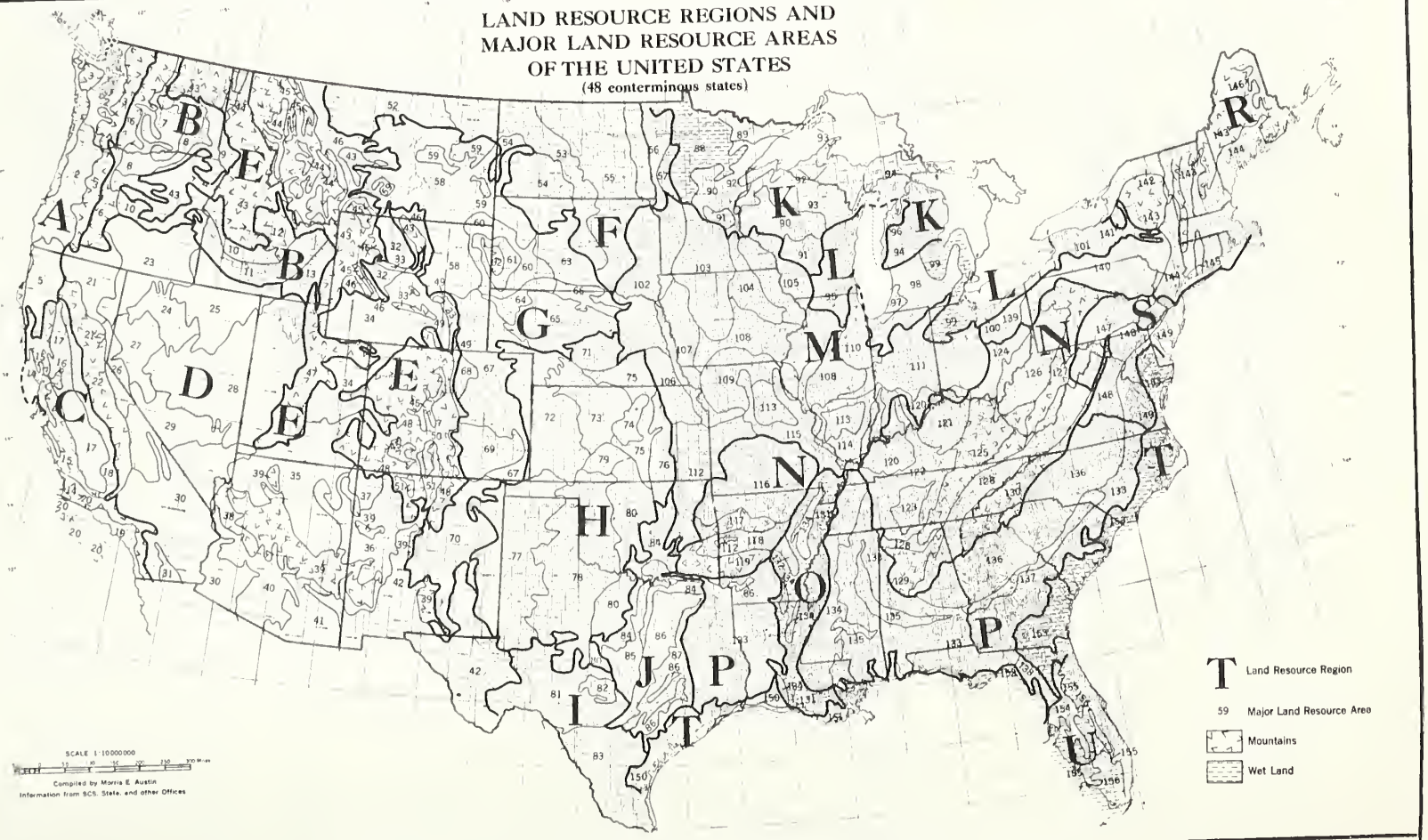
[1] Contribution from the Soil Conservation Service, U.S. Department of Agriculture.

[2] Agricultural engineer, Engineering Division, Washington, D.C.

LAND RESOURCE REGIONS AND MAJOR LAND RESOURCE AREAS OF THE UNITED STATES

(48 conterminous states)



LEGEND

A NORTHWESTERN FOREST, FORAGE, AND SPECIALTY CROP REGION

1 Northern Pacific Coast Range and Valleys
2 Willamette and Puget Sound Valleys
3 Olympic and Western Slope Cascade Mountains
4 California Coastal Redwood Belt
5 Siskiyou-Trinity Area

B NORTHWESTERN WHEAT AND RANGE REGION

6 Eastern Slope Cascade Mountaine
7 Columbia Basin
8 Columbia Plateau
9 Palouse and Nez-Perce Prairies
10 Upper Snake River Lava Plains and Hills
11 Snake River Plains
12 Lost River Valleys and Mountains
13 Eastern Idaho Plateaus

C CALIFORNIA SUBTROPICAL FRUIT, TPUCK AND SPECIALTY CROP REGION

14 Central California Valleys
15 Central California Coast Range
16 California Delta
17 Sacramento and San Joaquin Valleys
18 Sierra Nevada Foothills
19 Southern California Coastal Plain
20 Southern California Mountains

D WESTERN RANGE AND IRRIGATED REGION

21 Klamath and Shasta Valleys and Basins
22 Sierra Nevada Range
23 Malheur High Plateau
24 Humboldt Area
25 Owyhee High Plateau
26 Carson Basin and Mountains
27 Fallon-Lovelock Area
28 Great Salt Lake Area
29 Southern Nevada Basin and Range
30 Sonoran Basin and Range
31 Imperial Valley
32 Northern Intermountain Desertic Basins
33 Semiarid Rocky Mountains
34 Central Desertic Basins, Mountains and Plateaus
49 (See E below)
35 Colorado and Green Rivers Plateaus
36 New Mexico and Arizona Plateaus and Mesas
37 San Juan River Valley Mesas and Plateaus
38 Black, Hualpai, and Cerbat Mountains
39 Arizona and New Mexico Mountains
40 Central Arizona Basin and Range
41 Southeastern Arizona Basin and Range
42 Southern Desertic Basins, Plains and Mountains

E ROCKY MOUNTAIN RANGE AND FOREST REGION

43 Northern Rocky Mountains
44 Northern Rocky Mountain Valleys
45 Alpine Meadows and Rockland
46 Northern Rocky Mountain Foothills
47 Wasatch and Uinta Mountains
48 Southern Rocky Mountains
49 Southern Rocky Mountain Foothills
50 San Luis Valley
51 High Intermountain Valleys

F NORTHERN GREAT PLAINS SPRING WHEAT REGION

52 Brown Glaciated Plain
53 Dark Brown Glaciated Plain
54 Rolling Soft Shale Plain
55 Black Glaciated Plains
56 Red River Valley of the North
57 Western Minnesota Forest-Prairie Transition

G WESTERN GREAT PLAINS RANGE AND IRRIGATED REG.

58 Northern Rolling High Plains
59 Northern Smooth High Plains
60 Pierre Shale Plains and Badlands
61 Black Hills Footslopes
62 Black Hills
63 Rolling Pierre Shale Plains
64 Mixed Sandy and Silty Tableland
65 Nebraska Sand Hille
66 Dakota-Nebraska Eroded Tableland
67 Central High Plains
68 Irrigated Upper Platte River Valley
69 Upper Arkansas Valley Rolling Plains
70 Pecos-Canadian Plains and Valleys

H CENTRAL GREAT PLAINS WINTER WHEAT AND RANGE REGION

71 Central Nebraska Loess Hills
72 Central High Tableland
73 Rolling Plains and Breaks
74 Central Kansas Sandstone Hills
75 Central Loess Plains
76 Bluestem Hills
77 Southern High Plains
78 Central Rolling Red Plains
79 Great Bend Sand Plains
80 Central Rolling Red Prairies

I SOUTHWESTERN PLATEAUS AND PLAINS, RANGE AND COTTON REGION

81 Edwards Plateau
82 Texas Central Basin
83 Rio Grande Plain

J SOUTHWESTERN PRAIRIES, COTTON AND FORAGE REGION

84 Cross Timbers
85 Grant Prairie
86 Texas Blackland Prairie
87 Texas Claypan Area

K NORTHERN LAKE STATES FOREST AND FORAGE REG.

88 Northern Minnesota Swamps and Lakes
89 Minnesota Rockland Hills
90 Central Wieconsin and Minnesota Thin Loees and Till
91 Wisconsin and Minnesota Sandy Outwash
92 Superior Lake Plain
93 Northern Michigan and Wisconsin Stony, Sandy and Plains and Hills
94 Northern Michigan Sandy Drift

L LAKE STATES FRUIT, TRUCK, AND DAIRY REGION

95 Sôutheastern Wisconsin Drift Plain
96 Western Michigan Fruit Belt
97 Southwestern Michigan Fruit and Truck Belt
98 Southern Michigan Drift Plain
99 Erie-Huron Lake Plain
100 Erie Fruit and Truck Area
101 Ontario-Mohawk Plain

M CENTRAL FEED GRAINS AND LIVESTOCK REGION

102 Loess, Till, and Sandy Prairies
103 Central Iowa and Minnesota Till Prairies
104 Eastern Iowa and Minnesota Till Prairies
105 Northern Mississippi Valley, Loess Hills
106 Nebraska and Kansas Loess-Drift Hills
107 Iowa and Missouri Deep Loess Hills
108 Illinois and Iowa Deep Loess Hille
109 Iowa and Missouri Heavy Till Plain
110 Northern Illinois and Indiana Heavy Till Plain
(continued)
111 Indiana and Ohio Till Plain
112 Cherokee Prairies
113 Central Claypan Areas
114 Southern Illinois and Indiana Thin Loess and Till Plain
115 Central Mississippi Valley Wooded Slopes

N EAST AND CENTRAL GENERAL FARMING AND FOREST REGION

112 (See M Above)
116 Ozark Highland
117 Boston Mountains
118 Arkansas Valley and Ridgee
119 Ouachita Mountains
120 Kentucky and Indiana Sandstone and Shale Hills and Valleys
121 Kentucky Bluegrase
122 Highland Rim and Pennyroyal
123 Nashville Basin
124 Western Allegheny Plateau
125 Cumberland Plateau and Mountains
126 Central Allegheny Plateau
127 Eastern Allegheny Plateau and Mountains
128 Southern Appalachian Ridges and Valleys
129 Sand Mountain
130 Blue Ridge

O MISSISSIPPI DELTA COTTON AND FEED GRAINS REGION

131 Southern Mississippi Valley Alluvium
132 Eastern Arkansas Prairies
134 (See P below)

P SOUTH ATLANTIC AND GULF SLOPE CASH CROP, FOREST, AND LIVESTOCK REGION

86 (See J Above)
133 Southern Coastal Plain
134 Southern Mississippi Valley Silty Uplands
135 Alabama and Mississippi Blackland Prairies
136 Southern Piedmont
137 Carolina and Georgia Sandhills
138 North Central Florida Ridge

R NORTHEASTERN FORAGE AND FOREST REGION

139 Eastern Ohio Till Plain
140 Glaciated Allegheny Plateau and Catskill Mountains
141 Tughill Plateau
142 St. Lawrence-Champlain Plain
143 Northeastern Mountains
144 New England and Eastern New York Upland
145 Connecticut Valley
146 Aroostook Area

S NORTHERN ATLANTIC SLOPE TRUCK, FRUIT, AND POULTRY REGION

147 Northern Appalachian Ridges and Valleys
148 Northern Piedmont
149 Northern Coastal Plain

T ATLANTIC AND GULF COAST LOWLANDS, FOREST AND TRUCK CROP REGION

150 Gulf Coast Prairies
151 Gulf Coast Marsh
152 Gulf Coast Flatwoods
153 Atlantic Coast Flatwoods

U FLORIDA SUBTROPICAL FRUIT, TRUCK CROP AND RANGEREGION

154 South Central Florida Ridge
155 Southern Florida Flatwoods
156 Florida Everglades and Associated Areas

the SCS has provided technical assistance to date. For the most part, these structures were located on the more impermeable sites available. However, it is not always practicable to do this and have the structure where it will best serve the purpose for which it is installed. Also there are many farms and ranch units operating with inadequate water supply because they are located on soils unsuited in their natural state for impounding water.

The locations and extent of these problems and some of the remedial treatments that have been tried are given below in relation to Major Land Resource Areas and by groups of states as served by the SCS Engineering and Watershed Planning Units. Major Land Resource Areas are delineated on figure 1.

UPPER DARBY ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of Maine, New Hampshire, Vermont, Massachusetts, Connecticut, New York, Rhode Island, Pennsylvania, Maryland, Delaware, West Virginia, and Virginia. Pond construction in this group of States is at the rate of about 3500 annually. Records of the Soil Conservation Service show approximately 75,000 ponds on the land as of July 1, 1962.

Seepage losses from ponds can be quite serious in the Southern and Northern Appalachian Ridges and Valleys Resource Areas of Pennsylvania, Maryland, Virginia, and West Virginia. These areas are Nos. 128 and 147, respectively, on the map in figure 1. Seepage probably results from the flocculated residual soils over sink holes, cavernous and fractured limestone, and calcareous shales. Good seals have been obtained by filling existing voids with cinders, sand-gravel mixture or other coarse material, covering with compacted earth, then treating the entire reservoir area with polyphosphate at the rate of 1 pound per 20 square feet. The entire area is disked and recompacted as a final step. This method of sealing has been about 98 percent successful in these two land resource areas.

About 25 percent of the ponds constructed in the Ontario and Mohawk Plain and the Erie Fruit and Truck Belt Land Resource Areas of New York and Pennsylvania are likely to be affected by excessive seepage. These areas are Nos. 101 and 100, respectively, in figure 1.

Probable cause of leakage is the formation of vertical cracks in the flocculated, calcareous glacial till, permitting water to reach the underlying pervious zones of glacial till or fractured bedrock. Treatment with polyphosphate has been about 98 percent successful in these situations. Treatment with salt and calcium chloride was effective for awhile but the seal lasted less than 1 year. About 10 percent of the ponds in the New England and Eastern New York Upland resource area of Maine, New Hampshire, Vermont, Massachusetts, Connecticut, Rhode Island, and New York are likely to be affected by excessive leakage. This is area No. 144 in figure 1. The primary cause of leakage is the exposure of strata of clean sands and gravel during construction. Compacted earth blankets, sometimes treated with polyphosphate, have been about 90 percent successful. Bentonite at a rate of 1 pound per square foot has also been quite successful.

In the Southern Coastal Plain of Virginia, the Northern Coastal Plain of Massachusetts, New York, New Jersey and Maryland, and the Atlantic Coastal Flatwoods of Delaware and Virginia about one pond in five is likely to leak excessively. These are Nos. 133, 149, and 153, respectively, on the map. The usual cause of seepage is exposed strata of sand and

gravel. Treatment is with bentonite at a rate of approximately 1 pound per square foot if 0.005-mm. clay content is less than 15 percent, polyphosphate if clay content is higher than 15 percent.

There is a moderate possibility of excessive seepage from ponds in the Northern Piedmont area of Pennsylvania, Delaware, Maryland, and Virginia and the Southern Piedmont area of Virginia. These are Nos. 148 and 136, respectively, on the map. The usual cause is a thin mantle of silty residual soils over faults, fractures, and cleavage of bedrock. Compacted earth blankets treated with polyphosphate have given the best results in reducing leakage to an acceptable amount in these two resource areas.

SPARTANBURG ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of North Carolina, South Carolina, Tennessee, Georgia, Alabama, Mississippi, and Florida. Approximately 15,000 ponds and small irrigation reservoirs were constructed in the area in fiscal year 1962. Soil Conservation Service records show about 348,000 such structures in the group of States as of July 1, 1962.

Pond seepage is a very serious problem in the following Major Land Resource Areas:

- Kentucky Bluegrass in Kentucky and Nashville Basins area of Tennessee Nos. 121 and 123, respectively).
- Highland Rim and Pennyroyal area in Tennessee and north Alabama No. 122.
- Southern Appalachian Ridges and Valleys area of Tennessee, north Georgia, and north Alabama (No. 128).

Pond and reservoir sites in these areas are often underlain by cavernous limestone with thin mantles of soil cover. The degree of leakage depends upon the depth and textural qualities of the soil cover and the size and continuity of cavities. Every effort is made to avoid sites where conditions are unfavorable. In spite of this effort, a number of ponds develop serious seepage problems.

The major effort to develop a seal has been through the use of soda ash. After the pond bottom has been disked to a depth of about 8 inches, the sealing material is spread at the rate of approximately 1 pound per square yard and rolled with a wheeled tractor. This method is reported to have been successful in about one-half of the cases where it has been used. In the other half of the cases, partial successes to complete failures have been reported.

Improvements in this method or development of different methods will be necessary before sealing efforts in these land resource areas are considered successful. Other Major Land Resource Areas in the Southeast in which pond leakage is a lesser problem are:

- Southern Coastal Plain in southwest Georgia, southeast Alabama and north Florida (No. 133).
- Atlantic Coast Flatwoods in south Georgia and north Florida (No. 153).
- North-Central Florida Ridge (No. 138).
- Carolina and Georgia Sandhills in North Carolina, South Carolina, and Georgia (No. 137).

In these resource areas the problem stems from sandy, moderately permeable to permeable soils, underlain in most cases by limestone. Treatment with bentonite at the

rate of 1 to 2 pounds per square foot has been tried. This has been successful for relatively shallow ponds. As the depth of water increases, the effectiveness of the seal decreases.

Compacted sandy clay blankets have been used to a limited extent in various resource areas of the Southeast. The success of this method is dependent on foundation conditions and the adequacy of the blanket provided.

FORT WORTH ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of Arkansas, Louisiana, Oklahoma, Texas, and New Mexico. In this group of States approximately 14,000 ponds and small reservoirs are constructed annually with technical assistance from the Soil Conservation Service. About 320,000 have been constructed to date with SCS technical assistance.

Pond leakage is a very serious problem in the following Major Land Resource Areas:

- Pecos-Canadian Plains and Valleys in New Mexico (No. 70).
- Southern High Plains in New Mexico, Texas, and Oklahoma (No. 77).
- Southern Desertic Basins, Plains, and Mountains in Texas and New Mexico (No. 42).
- Central Rolling Red Plains of Texas and Oklahoma (No. 78).
- Rio Grande Plain in Texas (No. 83).

In Resource Areas Nos. 70 and 77 the sites that are underlain by caliche can be expected to have seepage problems on 50 percent of the desirable locations from a functional standpoint. These sites are avoided if there is a reasonable alternative for a water supply. In a few instances a natural seal has developed after a time where the inflow contained fine sediment. Compacted soil blankets have been used successfully in other cases.

Pond seepage problems in Resource Areas Nos. 42 and 78 are limited generally to areas of gypsiferous soils. Sealing methods that work on other soils have not been successful in these areas. It appears that nothing less than an impermeable lining will do the job in the gypsum soils. The granules, ledges, and boulders of gypsum provide adequate support for the weight of a lining and head of water if kept reasonably dry; otherwise, the gypsum dissolves, causing structural failure of the supporting foundation. Clay blankets and the other conventional kinds of treatment do not provide the needed protection against saturation.

Problems and solutions in Resource Area No. 83 are quite similar to those described above for Resource Areas Nos. 70 and 77. Leakage is caused by underlying strata or pockets of sand and gravel or caliche.

MILWAUKEE ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of Minnesota, Wisconsin, Michigan, Iowa, Illinois, Indiana, Ohio, Missouri, and Kentucky.

Annual construction of farm ponds in these States with technical assistance from the SCS amounts to approximately 10,000. Available information indicates something over 200,000 ponds have been constructed to date.

Major Land Resource Areas where leakage is a problem include:

- Northern Mississippi Valley Loess Hills in Wisconsin, Iowa, and Minnesota (No. 105).
- Iowa and Missouri Deep Loess Hills in Iowa and Missouri (No. 107).
- Southern Illinois and Indiana Thin Loess and Till Plain in Ohio (No. 114).
- Ozark Highlands in Missouri (No. 116).
- Kentucky and Indiana Sandstone and Shale Hills and Valleys in Indiana (No. 120).
- Kentucky Bluegrass in Kentucky and Indiana (No. 121).
- Highland Rim and Pennyroyal in Kentucky and Indiana (No. 122).
- Western Allegheny Plateau, Central Allegheny Plateau, Eastern Ohio Till Plain, and Glaciated Allegheny Plateau and Catskill Mountains in Ohio (Nos. 124, 126, 139, and 140 respectively).

In Resource Areas Nos. 105 and 107 the problem of pond leakage is most severe where the loess is deep or is underlain by sand or gravel and water table is not present. Both bentonite and polyphosphate have been tried in attempting to develop a seal. Neither of these materials has a fully satisfactory result.

The usual cause of leakage in Resource Area No. 114 is the presence of permeable soils in the reservoir area. Bentonite treatment has been tried in a few instances but with very little success. It is thought that poor application methods contributed to the failure.

Sandy soils, thinly bedded shale, and permeable glacial soils are blamed for pond leakage in Resource Area No. 124. Bentonite, soil cement, and polyphosphate have all been used as sealers. They have all been 80 to 95 percent effective when properly applied. However, the soil cement seal was later damaged by the trampling of cattle.

The underlying fractured limestone creates the leakage problem in Resource Area No. 126. No sealing tests have been made in this area. In fact very few ponds have been constructed in the area to date. Should the demand for ponds increase, it will be necessary to develop satisfactory sealing methods.

The problems in Resource Areas Nos. 139 and 140 are quite similar. Loss of water is through gravel lenses, porous stratified glacial till, and sandstone. No sealing tests have been made. However, it is thought that selected soil blankets, compacted and possibly chemically treated, will be adequate in most cases. The expense may be prohibitive if suitable soil must be hauled great distances for the blanket.

LINCOLN ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of Montana, Wyoming, Colorado, North Dakota, South Dakota, Nebraska, and Kansas. Farm pond construction in these Great Plains States averages approximately 10,000 ponds per year. It is estimated that 230,000 have been constructed to date.

Major Land Resource Areas in which leakage of ponds is a problem are:

- Central Deserti Basins, Mountains and Plateaus in Wyoming and Colorado (No. 34).
- Southern Rocky Mountains in Colorado (No. 48).
- Dark Brown Glaciated Plain in Montana, North Dakota and South Dakota (No. 53).
- Rolling Soft Shale Plain in North Dakota (No. 54).

- Loess Till and Sandy Prairies in Nebraska (No. 102).
- Northern Rolling High Plains in Montana, Wyoming, and South Dakota (No. 58).
- Pierre Shale Plains and Badlands in Montana, Wyoming, and South Dakota (No. 60).
- Black Hills Footslopes in Wyoming and South Dakota (No. 61).
- Rolling Pierre Shale Plains and Mixed Sandy and Silty Tableland in South Dakota (Nos. 63 and 64, respectively).
- Dakota-Nebraska Eroded Tableland in Nebraska and South Dakota (No. 66).
- Central High Plains and Upper Arkansas Valley Rolling Plains in Colorado (No. 67 and 69, respectively).
- Central Nebraska Loess Hills in Nebraska (No. 71).
- Rolling Plains and Breaks in Nebraska and Kansas (No. 73).
- Central Loess Plains in Nebraska (No. 75).
- Bluestem Hills in Kansas (No. 76).
- Loess, Till, and Sandy Prairies in Nebraska (No. 102).
- Cherokee Prairies in Kansas (No. 112).

Permeable materials, such as alluvium, talus, and sand, in reservoir areas and pond sites underlain by jointed bedrock, solution cavities, rock fractures, and coal beds are the conditions that appear to cause pond leakage in the major land resource areas listed above.

It is noticeable that in most resource areas a permeable reservoir area is combined with undesirable substrata or foundation conditions.

Selected clay blankets, placed in layers and compacted at near-optimum moisture, have provided satisfactory seals. Good initial success has been had with bentonite but leakage appeared to be increasing again after 3 to 5 years.

Polyphosphate has given good results where clay content of reservoir area was adequate. No evaluation as to lifespan has been made since first trials were made only a short time ago.

PORTLAND ENGINEERING AND WATERSHED PLANNING UNIT

This office serves the States of Washington, Oregon, Idaho, California, Arizona, Utah, Nevada, Hawaii, and Alaska.

Seepage from farm ponds and small overnight irrigation water storage reservoirs is a problem of some magnitude in all of the Western Region. The resource areas in which the problem is of greatest significance are as follows:

Major Land Resource Area	State
Columbia Basin (7)	Washington - Oregon
Columbia Plateau (8)	Washington - Oregon
Eastern Slope Cascade Mountains (6)	Washington - Oregon
Upper Snake River Lava Plains and Hills (10)	Oregon - Idaho
Eastern Idaho Plateaus (13)	Idaho
Sierra Nevada Foothills (18)	California
Fallon-Lovelock Area (27)	Nevada
Colorado and Green River Plateaus (35)	Utah - Arizona
Sonoran Basin and Range (30)	Arizona - California - Nevada
Southeastern Arizona Basin and Range (41)	Arizona
Hawaiian Islands	Hawaii

The usual causes of leakage are permeable soils or very shallow soils over fractured bedrock in the bottom and/or banks of the basins. In Resource Area No. 6 the seepage problem is associated with the widespread occurrence of a volcanic ash material. Coarse-textured, highly permeable soils are the principal cause of seepage in Resource Areas Nos. 7, 18, 27, and 30. In Hawaii the principal cause of seepage is the strongly aggregated clay soils that have extremely high coefficients of permeability.

Several methods of lining and sealing have been tried. Concrete and pneumatically applied mortar linings have been successful, but because of the high initial cost generally have been used only in small overnight irrigation reservoirs. Asphalt plank linings have been used successfully in Hawaii, both for stock-water ponds and small irrigation reservoirs. Their use has been limited due to the cost. Bentonite linings are probably the most extensively used method of sealing ponds and small reservoirs in the Western region, excluding Hawaii. These linings, when properly designed and applied, have been generally successful. Blankets of clay or other fine-textured soil have proven to be a successful method for small areas of exposed bedrock or other material.

SUMMARY

Water loss from farm ponds and small irrigation reservoirs by seepage is a serious problem in about 63 of the 156 Major Land Resource Areas in the 48 conterminous States. The extent of loss in the problem sites is such as to require some kind of lining or sealing for proper functioning of the structure.

The principal causes of seepage are:

- Permeable soils or strata of sand or gravel in the reservoir area.
- Shallow soils underlain by fractured bedrock, solution cavities, bedding planes, etc.
- Flocculated residual soils over cavernous and fractured limestone and calcareous shales.
- Gypsum content of the soil sufficiently high that significant voids develop as gypsum is dissolved.

Several methods of treatment have been tried with varying degrees of success.

- Compacted earth blankets containing clay have been very successful in sealing small areas of exposed, fractured rock or other permeable material provided there is ample structural support for the weight of the blanket and the impounded head of water. Chemical treatment of the blanket material may be needed to reduce permeability.
- Chemical treatment, usually polyphosphate, of the soils in the reservoir area provided clay content is above 15 percent. Excellent results have been obtained in the high-lime soils. More careful investigation is recommended as pH drops below 6.5. Usual application of polyphosphate for good site condition is 1 pound per 20 square feet.
- Treatment with bentonite at the rate of 1 pound per square foot has been successful in the moderately permeable sandy soils.
- Concrete and pneumatically applied mortar have been successful in Western States but high initial cost has so far limited this method to small overnight irrigation reservoirs.
- Prefabricated asphalt sheets or planks have been used successfully in Hawaii in both farm ponds and small irrigation reservoirs.

No satisfactory method of preventing seepage in the soils of high gypsum content has been developed. It appears that some type of lining that will keep the underlying soil material relatively dry will be needed. This is a serious problem in the affected areas since other sources of suitable water are scarce.

SEEPAGE LOSSES FROM NATURAL CHANNELS USED AS MAJOR CONVEYANCE FACILITIES[1]

John J. Vandertulip, Louis L. McDaniels, and Glenn D. Walker[2]

A severe 7-year drought in Texas during the 1950's, while causing widespread water shortages, financial losses, and considerable hardships, did provide an opportunity to obtain valuable basic data on water losses which occurred in natural channels. Records of streamflow during droughts and observations of dusty streambeds of water courses once considered perennial flowing streams emphasized to the water users and planners of water-use projects that firm demands for water could not be served by direct diversions without the development of conservation storage reservoirs.

The largest use of surface waters for irrigation and industrial purposes in Texas occurs in the lower reaches of the major river basins and along the Gulf Coastal Plains. Geographically, the topography of Texas generally affords the best reservoir sites in the middle and upper reaches of the major streams. The principal existing main-channel reservoirs capable of supplying lower river diversion demands have been developed in these middle and upper river reaches for almost all Texas streams. Experience in the use of these natural stream channels as conveyance facilities for the transport of water from these reservoirs to points of diversion downstream during periods of low flow has accentuated the necessity for determining the water loss and flow depletion characteristics of all channels so used.

Investigations of water losses in transport by streams in Texas, beginning with releases from storage in Lake Austin on the Colorado River in 1918, have provided some basic data for study and use as criteria in properly planning the development of upriver storage of water to be conveyed by natural channels to downriver water-use projects. These investigations have identified some of the essential considerations for proper planning which should include the following:

- Efficiency of operation with respect to flood inflows from intervening drainage area during periods of release from reservoirs.
- Natural encroachment of vegetation into stream channels and flood plains resulting from the reduction in the frequency of large scouring flood flows.
- Losses of released water to evaporation, transportation, seepage, and flood-plain storage.

Discussion of these considerations will be limited to the third item, although it, in part, is related to the second item, and at times to the first.

Seepage losses need to be considered in relation to flood-plain soils, geologic formations traversed, and whether or not stream channels have ground-water accretions from, or losses to, geologic formations. Data on streamflow are usually available for reservoir-

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[1] Contribution from the Texas Water Commission, Austin, Tex. Basic data obtained during the investigations described herein are contained in the Texas Water Commission Bulletin 5807 D, Channel Gain and Loss Investigations, Texas Streams, 1918-58, prepared in cooperation with the U.S. Geological Survey, Austin District.

[2] Chief engineer; head, Hydrologic Research Program; and hydrologist; respectively.

operation studies, but special measurements of streamflow are needed in seepage investigations to delineate channel characteristics by reaches of a stream. In many instances, complete ground-water investigations and studies may be required to supplement surface-water investigations in order to determine these loss or gain characteristics.

Nonbeneficial consumptive uses of water in some areas are exacting a heavy toll of surface-water resources. These uses consist largely of water demands of nonbeneficial vegetation plus losses incident to evaporation from swamps and flood-plain storage and badly deteriorated stream channels. Much of the water loss from stream channels is originally a seepage loss which is subsequently intercepted by vegetation and transpired. In many places, the surface areas covered by this vegetation are increasing. Upstream reservoir construction and operation have resulted in substantial reductions in the frequency of large flood discharges, with a resulting natural encroachment on the channel by phreatophytes and other undesirable vegetation. As a result of such encroachment, channel discharge capacities are decreased. This then results in more frequent overbank flow conditions and additional losses of flood flows. As channel capacities are decreased and flows are restricted, the "pumping" of water by flood-plain vegetation effects a reverse gradient away from the channel which causes an increase in the seepage rates and losses. The Colorado River below Austin provides an example of the decrease in channel discharge capacities resulting from such channel deterioration and modification of flood flows. Since 1940 when operation of Lake Travis began, upstream floods on the Colorado have been harnessed. As a result during the 22-year period to 1961, channel deterioration downstream caused an increase of 6 to 8 feet in river stage to pass flows ranging from about 40,000 to 100,000 cubic feet per second at points 106 and 144 miles downstream.

The most notable example of nonbeneficial consumptive use, for which there is some hope of relief through remedial measures, is the increasing areas on Texas streams covered by a dense growth of saltcedars. This growth has resulted in a substantial increase in the losses of reservoir releases through the stream channel to downstream diversion points. Figures 1 through 5 show the encroachment of saltcedar at two locations on the Pecos River.

Studies have been made by the Red Bluff Water Power Control District, and by the U.S. Bureau of Reclamation on losses of water released from Red Bluff Reservoir to the downstream irrigation districts along the Pecos River. About 82 percent of these releases are made during the 5-month period from April through August. An excellent correlation exists between the river conveyance losses and the volume of the releases. These analyses were made without respect to time. It was found that the losses were indirectly proportional to the releases--the larger the release the smaller the ratio of loss, as indicated in the following table. The losses of releases from Red Bluff Reservoir to Pecos River, between dam and diversion points were as follows:

Releases	Losses
(Acre-feet)	*(Percent)*
120,000 to 60,000	20
60,000 to 10,000	20-40
10,000 to 1,000	40-60

Figure 1.--Vegetation in the vicinity of the streamflow gaging station, Pecos River near Orla, Tex., April 3, 1940.

Figure 2.--Saltcedars in the vicinity of the streamflow gaging station, Pecos River near Orla, Tex., September 14, 1961, showing the growth during the 21-year period from 1940.

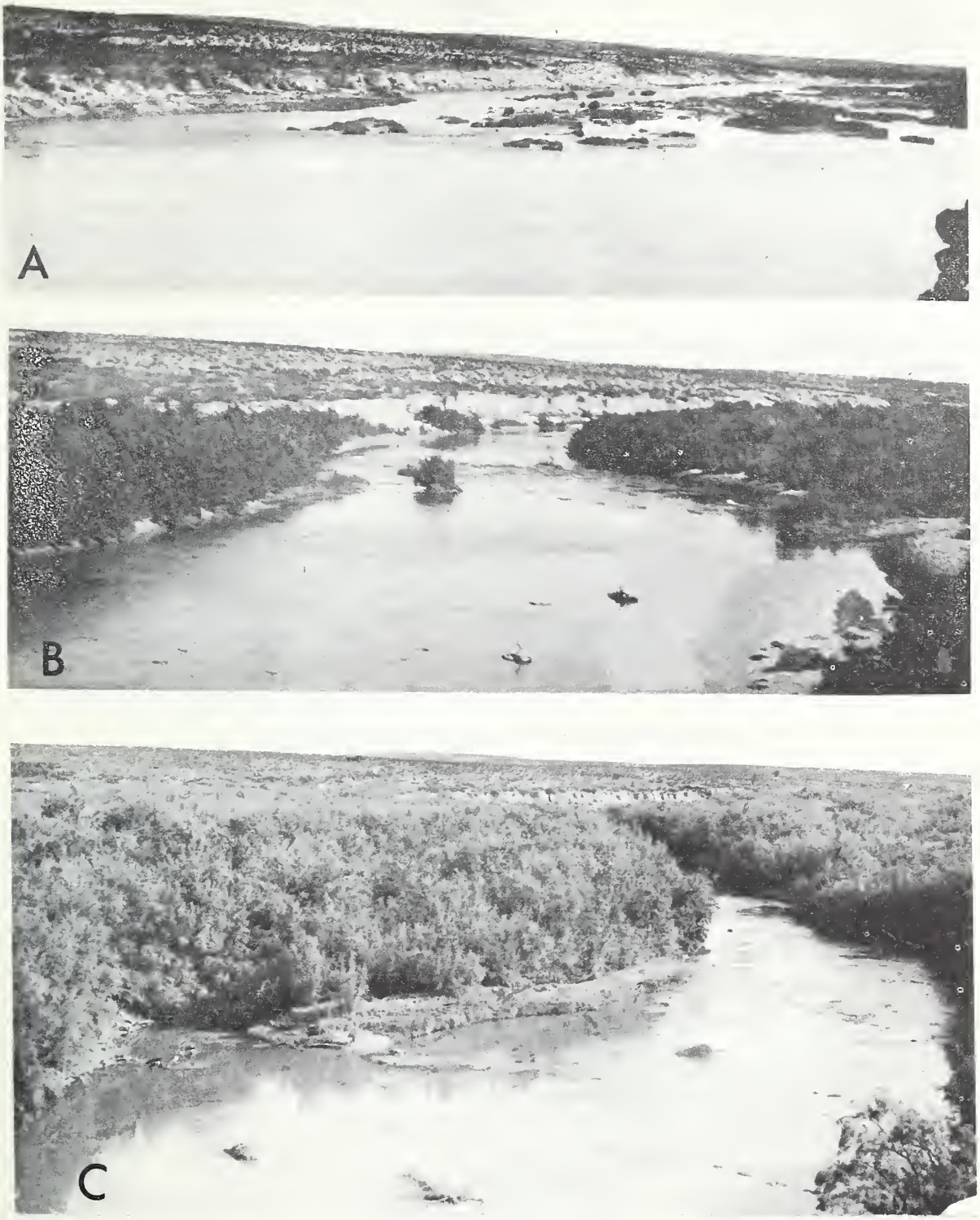

Figure 3.--Vegetation in the vicinity of the discontinued streamflow gaging station. This channel reach is about 2 miles north of the Texas-New Mexico state line and in the upper end of Red Bluff Reservoir, Pecos River near Angelos, Tex.; photographed: A, Prior to 1918 and probably after 1914; B, November 1936; and C, October 1961.

LOW-FLOW INVESTIGATIONS

Since 1918, nearly 150 low-flow investigations have been made to determine quantitative changes through long reaches of natural stream channels. The investigations were made to show gains or losses during a period of base flow; that is, when the total flow of the stream was contributed by springs or seeps from aquifers with no direct runoff from recent storms. Studies have ranged from a reconnaissance-type study, with a few discharge measurements of main-stream flow, to comprehensive types with many measurements of main-stream flow, tributary inflow, and diversions. Recent comprehensive investigations have consisted of current-meter measurement of flow as much as possible and extensive notes made on bank seepage, springs, channel conditions, bed composition, and vegetation in the streambed, on the banks, and in the stream valley.

The data in the early investigations are especially valuable, having been obtained before major river developments took place, and generally represent natural conditions.

Low-flow investigations are a regular part of the basic-data program of the Commission. These data are important to both ground-water and surface-water studies. Increases in ground-water pumping often gradually reduce the base inflow to streams and can result in reversals in hydraulic gradient with streams losing water to the ground-water formation. When large losses occur in reservoir releases, detailed low-flow investigations are undertaken to isolate principal-loss sections. Complex geo-hydraulic conditions often require detailed surface, ground, and quality of water studies. A 3-year intensive study of the Brazos River between Whitney Dam and the mouth is now being conducted by the U.S. Geological Survey for the Commission. This study covers 439 river miles and areas in 12 countries.

DELIVERY OF WATER INVESTIGATIONS

Delivery of water investigations in Texas have been made during periods when water was being released from reservoirs and allowed to flow down natural stream channels to points of diversion or use. These investigations provide information on losses encountered in conveying water downstream, time of travel of released water, peak-flow reductions, and changes in rate of flow of released water as it progressed downstream. This information is essential to enab' the water user to compute the rate of release that will effect maximum recovery of released water at the points of diversion or use.

Generally, these investigations were made when stored water was being released to meet water demands during drought periods. Channel reaches used for conveyance were usually dry or nearly dry; consequently, bank storage prior to release was at a minimum. Water was lost in varying amounts through evaporation, transpiration, and seepage or recharge to ground-water reservoirs when the geology favored such a loss.

Lake Austin to Bay City

The first investigation was made during the 1918 drought. A special release was made from Lake Austin down the Colorado River to irrigators in the vicinity of Bay City, Tex. The irrigators requested the Texas Board of Water Engineers (now the Texas Water Commission) to use its statutory authority to insure that the released water was undisturbed through the natural course of the Colorado River to the irrigators. Hence, the first channel-loss investigation was made.

Data obtained indicate 25,300 acre-feet of water were released from Lake Austin to the Colorado River, which had been in a falling stage for several days prior to the release. Release was made from July 5 through July 28, 1918. As of August 6, the Board of Water Engineers representative estimated that 75 percent of the release (19,034 acre-feet) had passed Columbus, 163 miles downstream. By August 7, 60 percent (15,118 acre-feet) had passed Wharton, 231 miles downstream. The total quantity received by the irrigators (256 miles below Lake Austin) was slightly less than that which passed Wharton due to losses in "The Raft"--debris and logjam--and unrecovered flow through the "East Channel" of the river at Bay City. Time of travel of first water was about 5 days to Columbus and 7 days to Wharton.

Brownwood Reservoir to Bay City

The next investigation, made in 1934, recorded the movement of purchased water released from Brownwood Reservoir into Pecan Bayou, thence down the Colorado River to the Bay City Water Co. The first water was released June 27, 1934. A total of 10,600 acre-feet was released through July 2, 1934, of which 56 percent was lost between Brownwood and Eagle Lake (432 river miles) and 70 percent was lost before reaching Wharton (477 river miles). It is assumed no significant losses occurred between Wharton and Bay City.

The reservoir gates were opened again on July 3 to release another increment of the purchased water. Due to an accident to their mechanism, the gates could not be closed and the reservoir was drained of 71,800 acre-feet of stored water as measured at the Brownwood gaging station. The rate of release varied as numerous attempts were made to close the gates. Of the first 65,900 acre-feet released, 33 percent was lost before reaching Eagle Lake, of which 11,800 acre-feet was lost above Austin (246 river miles) and 10,100 acre-feet was lost below Lake Austin.

Neither discharge nor rainfall records indicate any surface runoff prior to July 22 that would affect the accuracy of the study of losses of released water. For over a month prior to releases, there was no rain nor rise of any significance in any part of the stream between Brownwood and Bay City. The appearance of flood water in the Colorado at the San Saba gage on July 22, and at later dates downstream, made the loss study for the total release impossible. However, it is considered that the 65,900 acre-feet of water released during the period June 27 to July 20, could be followed through to Bay City with fair accuracy.

Red River to Lake Dallas

As an emergency measure in 1953, the city of Dallas constructed a pumping station on Red River directly north of Gainesville for the purpose of diverting water over the Red-Trinity River divide and into Lake Dallas for municipal use.

Six electric pumps (delivering 19 cubic feet per second each) diverted water to the head of Pecan Creek--a tributary to Elm Fork Trinity River. A channel-loss investigation of this operation was made from February 10 to March 3, 1954. The following considerations were agreed to as the method of pump operation:

- The interval-of-flow method be used to determine losses, with an interval of 72 hours' pumping and 72 hours' shutdown or recession time between runs.
- Three rates of flow be investigated--flow from 6, 4, and 2 pumps.
- Six pumps be started at noon February 11.

During this period, 1,363 acre-feet of water was pumped into the head of Pecan Creek 3.5 miles above Gainesville. Of this amount 1,272 acre-feet of water reached the head of Lake Dallas 33.9 miles from the pumps (table 1).

Table 1.--Pertinent data on water delivery from Pecan Creek to Lake Dallas, Feb. 11 to Mar. 3, 1954

Gaging point	Distance below pump station	Gaged flow, pumps operating 72 hours			Total
		6 pumps	4 pumps	2 pumps	
	Miles	*Acre-feet*	*Acre-feet*	*Acre-feet*	*Acre-feet*
Head of Pecan Creek.....	3.0	685	454	224	1,363
Elm Fork Trinity River, 6 mi. south of Gainesville.....	12.6	683	421	220	1,324
Elm Fork Trinity River, 10 mi. northeast of Denton.....	33.9	645	421	206	1,272
Total loss of water:					
Acre-feet.....		40	33	18	91
Percent.....		5.8	7.3	8.0	6.7

The weather was ideal for this type of investigation because no rain fell in the vicinity and the sun was shining much of the time. Conditions were conducive to high evaporation and transpiration, and losses should have been near the maximum for a winter season. The percentage loss in this instance is misleading as the river channel distance is much shorter than that of other examples given herein. When compared on the basis of a percent-loss per mile of channel distance, these losses were quite large.

Whitney Reservoir to Richmond

From August 1 to August 31, 1954, 47,700 acre-feet of water was released from Whitney Reservoir into the Brazos River channel for use by irrigators below Richmond. Of this release, 76 percent (36,200 acre-feet) arrived at Hempstead 246 miles downstream from Whitney Dam, and 69 percent (33,100 acre-feet) arrived at Richmond 349 miles downstream. Travel time was 137 hours to Hempstead and about 193 hours to Richmond. At the time of this release, the river channel was wet because of residual water from previous releases, thereby reducing expected initial losses.

About 9,500 acre-feet (65 percent) of the 14,600 acre-feet lost in transit from Whitney to Richmond is attributed to evaporation as computed with 9.0-inch evaporation rate and 300-foot average river width. These results can be expected only when conditions are similar to those existing during the period of this investigation.

A second investigation was made on this same reach during the period July 2 to August 5, 1956, when 103,000 acre-feet of water was released for irrigators below Richmond. Of this released water, 71,300 acre-feet, or 69 percent, arrived at Richmond. The time of travel of first water released was about 198 hours.

No initial loss occurred, as previous releases had wetted the channel and saturated the alluvium beds in the reach. Tributary inflow and runoff from rainfall were scant during the period. Table 2 lists evaporation losses before the releases reach gaging stations.

Table 2.--Loss of 103,000 acre-feet released by reaches between gaging stations

Gaging station	Water arriving	Released water	Lost in transit	Lost between gages	Travel time of first water	Distance below Whitney Dam
	Acre-feet	*Percent*	*Acre-feet*	*Percent*	*Hours*	*Miles*
Waco.....	94,900	92	8,100	8.0	18	38
Bryan.....	79,400	77	15,500	15.0	90	157
Hempstead.	74,500	72	4,900	5.0	140	246
Richmond..	71,300	69	3,200	3.0	198	349

About 15,000 of the 31,700 acre-feet lost in transit is attributed to evaporation. This loss was estimated on the basis of 349 river miles, averaging 325 feet in width, with an evaporation rate of 11.32 inches during the period of 1.2 months.

The transit losses and travel time for this investigation agree very closely with those for the investigations of August 1 to 31, 1954. Although about twice the volume of water was released in 1956, it should be noted that in both cases the channel bed and banks were wetted by previous releases of water that materially reduced channel losses other than evaporation from water surfaces.

Belton Reservoir to Richmond

Beginning November 1 and ending December 14, 1956, 73,000 acre-feet of water was released from Belton Reservoir for conveyance down the channels of the Leon, Little, and Brazos Rivers for industrial use in the vicinity of Freeport.

The release water traversed 342 miles to Richmond in 202 hours. Analysis of stream-flow records show that about 54,300 acre-feet of the released water reached Richmond. The loss (18,700 acre-feet) was 25.6 percent of the initial release.

This investigation was complicated by a considerable amount of flood inflow resulting from rains soon after release of water began and at the end of the period. Results are considered at the best as "fair."

A detailed explanation of each of the above investigations and others which have been made is beyond the scope of this paper. However, several similarities can be noted:

- Losses, both in volume and as percentage of release, increased with an increase in channel distance.
- Losses in most instances were in excess of that amount which could be attributed to evaporation from water surfaces.
- When reservoir releases are made intermittently, the losses in the natural channel can be expected to be of the order of one percent of the volume of the release for each 8 miles of river channel.

SUMMARY

Data which are available support the contention that if a reservoir or reservoirs for a water-use project are to be located considerable distance upstream from a point or points of diversion and if stored water is to be conveyed thereto through natural river channels, the factor of the quantity of water lost from the channel may be the governing factor in the success or failure of the project. Total losses during drought periods for long channels often exceed 10 percent of the water released. Any assumptions made in hydrologic studies of a river system with relation to such proposed projects should be examined very carefully with respect to assumptions concerning conveyance losses. In many instances, detailed studies of ground-water and surface water conditions are warranted before proceeding with construction of water-use projects which are usually multimillion dollar undertakings.

SEEPAGE PROBLEMS IN THE JET AGE[1]

H. Shipley[2]

One day Lloyd Myers called up and suggested that I talk on the subject of the Jet Age with Seepage Problems for the Symposium. I told him that I would accept the assignment and thought nothing more of it. However, upon receiving the preliminary report of the program, I realized he was in earnest.

I purchased a text that one of the professors at Arizona State University recommended. As I looked through the book and tried to read portions of it, the complex equations were impressive. Turning to the index, I found 'seepage' listed on page 291 and would like to read a portion of the text:

> "The following general assumptions relating to reservoirs were made. (1) No allowance was made for maintaining permanent pools, recreation, fish culture and the like. (2) Loss of storage by sedimentation was not taken into account although this is a basic problem in the arid and semi-arid portion of the United States. (3) The hydrologic factors of evaporation and seepage from reservoirs were neglected because the work involved in searching out reasonable information and incorporating it into the simulation seemed disproportionately greater than the benefit to be derived."

Gentlemen, I would like to straighten out some of these points.

No. 1. Fish and Recreation.--This is an item regardless of whether the engineer figures it into his system or not, it is very much there. As a good example, I can illustrate with the Clear Creek Reservoir. This reservoir was designed by Phelps Dodge on a Little Colorado tributary and will have a capacity of 14,000 acre-feet. Immediately upon the announcement that Phelps Dodge was considering the construction of such a dam, the

[1] Contribution from the Salt River Project, Phoenix, Ariz.
[2] Associate general manager.

Fish and Wildlife people appropriated some 3,000 acre-feet for fish culture and recreation. So, hereafter let this be a warning to anyone considering the construction of a reservoir system or a pond of water. They had better include a permanent pool or dead storage for recreation and fish culture, which is a very important factor in today's life of the American people.

No. 2. Sediment.--Considered a major problem in the West. This is very interesting because when the Roosevelt Dam was constructed, the life of this dam was predicted to be 50 years since by then, it would be all silted up. However, the silt problem has been such a small factor that we no longer bother to take sedimentation surveys in less than 15-year periods. I am not saying that sedimentation is not a problem, but it should be carefully studied before it is established that it is a major problem.

No. 3. Seepage.--The text stated that this was an important factor. Let me illustrate how critical this little seepage problem can get. In 1960, the city of Williams decided that they needed another reservoir system due to the fact that their existing reservoir system was insufficient to provide a water supply for the existing population and the population that would be settling in the next 10 to 20 years. Since their reservoir system is located on the drainage of the Little Colorado basin, they proceeded to stake out a dam site on the Verde system, on a tributary that was appropriated by the Salt River Project back in 1902.

Naturally we here on the Salt River Project were quite concerned and immediately made an on-the-spot survey. It became very obvious to those who were there that this was strictly a recreational promotion, and, in fact, it was stated that when the need became critical for domestic water, a pipeline would be constructed to this reservoir. However, the reservoir was needed immediately. We also analyzed a consulting engineer's report regarding the water system of the city of Williams.

Of the 740 million gallons of water in the city of Williams reservoir in 1958, it was estimated that 12 percent, or 90 million gallons, was sold; 20 percent, or 148 million gallons, evaporated; and 68 percent, or 502 million gallons, disappeared as seepage. The city has recognized that seepage is a problem.

Now, gentlemen, I would like to evaluate the seepage problem right here on our Salt River Project.

From figure 1 it is very obvious that seepage is a prime factor and considerable effort is being made to combat this wasted water. Working very closely with Lloyd Myers and his Agricultural Research Services staff and with Cliff Pugh of the local office of the Bureau of Reclamation and his staff, we are striving to develop a low-cost lining. We on the Project feel that this should be the Number 1 research assignment, to reduce seepage in the many unlined irrigation laterals throughout the western states. The reason we feel so strongly regarding this concept is that with the assistance of the Phoenix division of AiResearch a method has been developed to provide the necessary maintenance (removing silt and moss) from concrete-lined ditches. The present mechanical methods would destroy a low-cost lining and we envision the use of this AiResearch compressor for maintaining such linings. This piece of equipment will be demonstrated in operation Thursday morning to clearly illustrate the potential of this unit. It will also bring us closer to the original assignment--Seepage Problems in the Jet Age. By the way, we on the Project refer to this particular equipment very affectionately by the name of the "Blowin' Goose."

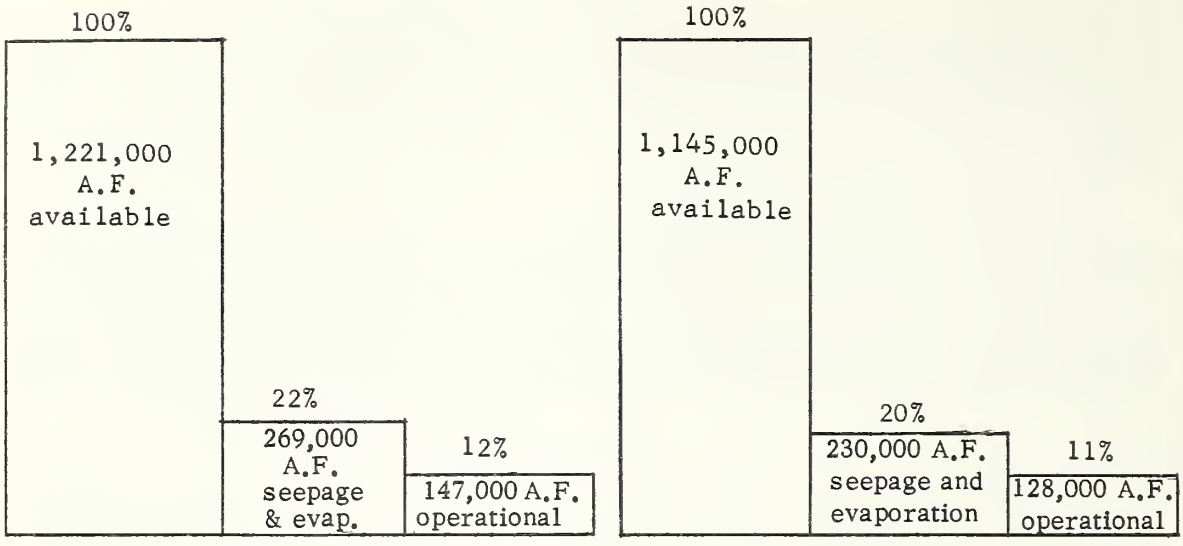

Figure 1.--Results of Salt River Project Water Conservation Program.

[Slides were shown to the group to familiarize them with the Salt River Project and also to illustrate some of the methods used to reduce seepage here on the Project. Most important--the methods to measure the water as accurately as possible for delivery to the land and to determine the reaches of the irrigation system where the seepage is considerable.]

[The final slides depicted the various stages of development of the irrigation here in the valley up to the present Jet Age.]

DISCUSSION

Tuesday morning - First Session

Wilder:

Mr. Coyle spoke about reservoir linings and quite correctly referred to the relatively high cost of concrete and shotcrete linings for reservoirs. I would like to throw in the comment that in Southern California quite a few irrigators are lining storage reservoirs with a relatively thin layer of compacted soil cement. This is not to be confused with concrete. It's not of the same quality, but in climatic conditions such as we have in this area and in Southern California, I believe this is a solution which ought to be considered.

This has some rather obvious and interesting advantages in that the irrigator with his regular help can do the job quite satisfactorily, using a minimum of rented or borrowed equipment and some help from a SCS engineer who has seen this done, or on occasion from some of our people. I think this is something that might be considered, certainly in the mild climates of Southern California and Arizona, and also, with a few more precautions and perhaps a little higher cement content, in some of the more rigorous climates.

Pugh:

I wonder if you could clarify for us what help the Portland Cement Association itself has to offer. I think we are all familiar here with the SCS's aid.

Wilder:

The Portland Cement Association has available some rather simple mimeographed sets of directions for doing this job including suggested layouts and procedures, and these can be made available on request to anyone who is interested. In some areas we have a field man who has had some experience with this who can consult with the irrigator, or whoever is going to do the job, and give him some suggestions and perhaps some help in getting the work started. I suggest that if someone has an idea of doing this, the best way of finding out what we can offer in his particular area is to contact one of our district offices, or our Chicago headquarters, or me in Los Angeles.

Slotta:

Mr. Shipley, with regard to the ground water-level drop--the level dropped from 0 to 220 feet since 1920--the question is: How far do you expect that you will be able to go and still be able to pump? Another question directed along this line is: How long or how far of a distance do you anticipate--or do you know--that this water has come? Has it been dated? Have you used the carbon date treatment on this? And then, finally, how long will you be able to continue this study?

Shipley:

I think Mr. Pugh is better qualified to answer it than I am. I believe he is making a study of the water situation. But I can mention about the pumping. I happened to be in the Bakersfield area about 17 years ago, and they were pumping just below Tehachapi in the Bakersfield area, then at 450 to 500 feet--so, to answer your question, whatever the economy can stand. In fact, right now the University of Arizona is making a study on that very same item--on the cost of pumping and how deep you can go. That is strictly an economic factor; just what can you support? We have a very peculiar situation here. We subsidize the cost of water because if we didn't, we couldn't compete with areas that have access to greater amounts of water, and less expensive, but that isn't answering your question about carbon dating the water. I think U.S. Geological Survey (USGS) is probably working on something like that. We, as an operating organization, do not do that type of work, although I think it is very interesting. We do know, as Mr. Pugh mentioned, that from this reservoir area in the Salt River Valley, it's going out about twice as fast as it's coming in, so wherever it's coming from, it isn't coming in fast enough.

The Bureau of Reclamation is making an intensive ground water study of this whole area in cooperation with the USGS, in which we are trying to define the extent of the ground water basin to find out its geographic limits. We mean to find out what volume of water there is here, and what is the permeability of the aquifer, and so forth. As for the question regarding how far we can afford to pump, that is an economic problem; it depends on the price of cotton as much as anything else. I might mention that the Salt River Project that Mr. Shipley has been talking about has about the best water situation in this whole area. Some people in adjacent areas are pumping up to 600 feet right now. The static ground water levels are down to 400 feet plus in many of these outlying areas, and their water table is not dropping 10 feet per year; it is dropping 20 plus feet. Also, there is subsidence going along with it. I don't think that is noticeable right here, but in the outlying areas, there is subsidence that is becoming noticeable as the ground water is withdrawn.

DAMAGE TO LANDS BY SEEPAGE[1]

John G. Sutton[2]

Damage to lands by seepage is a widespread and serious problem, especially in the western irrigated areas. For the purposes of this paper I will confine the discussion primarily to land damage from seepage from canals, reservoirs, and farm ponds where there is opportunity to reduce damage by lining or other seepage control treatment.

In the humid area of the United States there are large areas of land damaged by seepage from hillside or from adjoining streams where the damage may be reduced by appropriate drainage works. In the humid area seepage from ponds and reservoirs is relatively minor in extent, and the loss of water from such structures is of greater importance. Occasionally, however, small areas below dams are damaged by seepage. Substantial areas of flood plain land are also damaged by seepage from navigation and pools of other dams in major rivers. Often these land damages are seasonal in nature. However, remedial measures on flood plain lands in the humid area generally are confined to levees, subsurface drains, and pumps.

In thinking of land damage by seepage in the western irrigated areas perhaps the first question we may ask is the extent of the problem. The National Inventory of Conservation needs (8)[3] has determined the areas of land where excess water is the dominant problem, the areas where excess water is the secondary problem, and the areas needing treatment. In the 17 Western States there are some 31 million acres of land irrigated on farms according to the 1959 census. Of this total the Needs Inventory shows that on 5.5 million acres the dominant problem is excess water and on 1.8 million acres, excess water is classed as the secondary problem. Thus excess water is a problem on over 7 million acres of irrigated crop land or almost one-fourth of all irrigated land in 17 Western States.

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[1] Contribution from the Soil Conservation Service, U.S. Department of Agriculture.

[2] Drainage engineer, Engineering Division, Washington, D.C.

[3] Underscored numbers in parentheses refer to Literature Cited at end of this paper.

The large losses which occur in conveying the water in canals to the farm and through deep percolation losses on the farm provide an explanation of the extensive problem of excess water on these irrigated lands. The U.S. Geological Survey (9) estimated that in the 17 western irrigated States the conveyance losses in delivering the water from reservoir and streams to the farm amounted to 25 million acre-feet and the USGS also estimated that 89 million acre-feet was delivered to farms in the 17 Western States. The Soil Conservation Service has estimated in its report to the Senate Select Committee on Water Resources (6) that the average farm irrigation efficiency is about 47 percent. This would mean that of the 89 million acre-feet delivered to farms in the 17 Western States 47 million acre-feet was handled by farms without benefit. Much of this water was lost in farm conveyance and through percolation losses below the root zone. These water losses which grossed 72 million acre-feet resulted in the 7-million-acre wetland problem in the western irrigated States.

In comparison of the above figures it should be noted that the estimated gross farm losses of 47 million acre-feet is almost twice the 25 million acre-feet conveyance losses of water to the farms. These figures explain why so many drainage problems encountered on farms are complex in solution and need a combination of improved water management, canal lining or other seepage control measures, and the installation of subsurface drains for solution.

In the western irrigated areas the rise of the water table due to seepage generally brings to the surface saline or alkali salts. These salts damage land and crops. In many cases the land damage due to harmful salts is permanent or at least expensive to correct. Probably this is the most serious aspect of land damage due to seepage in western irrigated areas.

It has been estimated that in the 17 Western States approximately one-third of the irrigated and potentially irrigable lands is affected to some extent by saline and alkali problems. General observations also indicate that there is a very close relationship between high water table and the rise of saline and alkali salts. In fact, the descriptive terms "salty land," "wet land," "land with high water table," etc., generally apply to the same area on most irrigated farms.

There is another factor which complicates the planning of drainage and remedial works. In order to prevent the salts which are in the soil from rising to the surface it is necessary to apply more water than required by the crops. This minimum excess application may be 10 to 25 percent of irrigation water applied to the land depending on site conditions. Also, more deep percolation losses may occur at the upper or lower ends than at the middle of a field irrigated by surface methods. Thus, even with the best of water management some deep percolation losses will occur, and on many farms high water and salinity need to be controlled by subsurface drains. However, the irrigation engineers have found that large savings in water may be accomplished through proper water management as well as through seepage control by canal lining and installation of pipes to convey water. Generally when wetland problems occur in one area, the solution includes a combination of better water management, seepage control, and drainage.

At this point we may consider the more detailed aspects of land damage due to seepage and possibly consider in some detail what must be done to correct such damages. It would greatly simplify the problems if there existed a simple relationship between the application of seepage control methods and the effects on land damage. However, this relationship is not simple as the source of the damaging water may include seepage from a main canal, a farm distribution ditch, and deep percolation below the root zone. The seepage water

from canal and farm ditches usually mix with the water percolating from irrigated fields and so it loses its identity. This is especially true of water that moves laterally from its source to the point of land damage. Canal leakage at considerable distance may be the primary cause of land damage.

In examining areas which have suffered land damage due to high water table and where it is desirable to install corrective measures, there are some general principles which apply. Adequate investigations should be planned and carried out over a sufficient period of time to determine the worst ground water conditions and to plan adequate remedial measures. Nearly always the investigation and the remedial measures must be tailored to fit the site conditions. All aspects of the water damage, water supply, salt, soil, and cropping conditions must be considered.

The modern methods of making drainage investigations (7) by the Soil Conservation Service provide information on the source of damaging water, the path through which it moves, and criteria to plan the remedial measures. These investigations may include the installation of piezometers in the soil to measure the hydrostatic pressures in the soil and the direction of ground water movement. Piezometers are often installed in batteries of three, say at 4, 8, and 12 feet or at other required depths.

Figure 1, taken from the 1947 Imperial Valley Report (3), shows results of piezometer readings on the Meloland Experiment Station. At each location in the experiments four piezometers were put down and terminated at depths of 4 or 5, 7, 10, and 14 feet below the surface. The figures show the readings of water level below the datum plane for the piezometer terminating at the location shown. Since the lowest readings reflect the highest hydrostatic pressures, the ground water moves from the areas of low readings to areas of

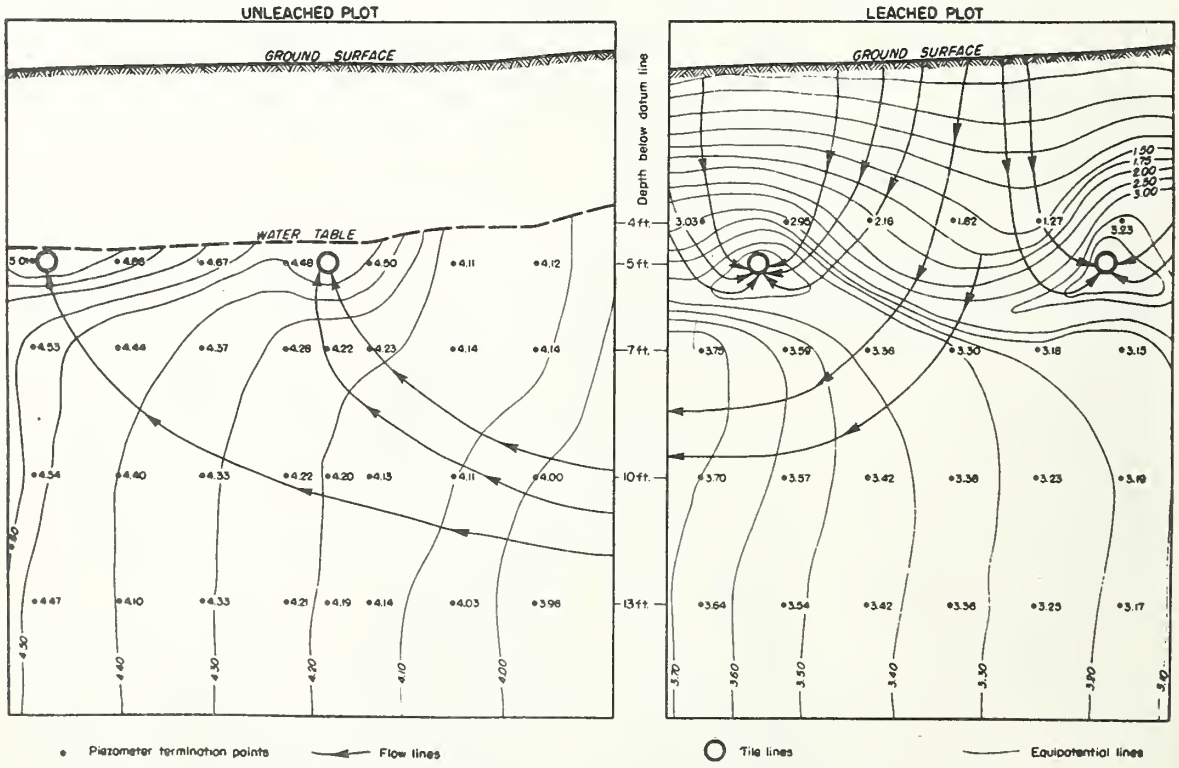

Figure 1.--Streamline flow experiments at Meloland Experiment Station.

high readings. Equipotential lines were drawn through points of equal hydrostatic pressure and are shown in figure 1. Flow lines were drawn at right angles to the equipotential lines.

A chart of this kind is a valuable tool in drainage investigations to determine the hydrostatic pressures and the direction of subsurface flow. Where investigations of this kind are made at the site of a leaky canal they may pinpoint the ground water flow from the canal. Figure 2 shows how a battery of piezometers may be used to detect the direction of ground water movement.

Another phase of drainage investigations is the study of observation wells to determine variations in depths to water table throughout the season. The height of the ground water table is determined at a critical period during the irrigating season. Such studies indicate the area of land damaged by high water table, and the direction of flow of the ground water.

Results of such studies may be shown on a working map as illustrated in figure 3. This map was prepared from a contour map, and ground water contours were taken during a critical period of high water. Readings should be made at intervals during the year and at least one set selected as being critical. Depths of water table were computed from ground elevations and water table elevations plotted on working maps. Finally a cross-hatched map to show areas of high water was completed as shown in figure 3. The map indicates the fan-shaped wet area is due to canal seepage. This is substantiated by the fact that there is no wet land above the canal, that the wet land is adjacent to and below the canal and the direction of ground water flow. Studies of the irrigation canal losses would then be in order to determine the feasibility of lining or seepage control measures to be installed. It would be expected that this wet area would be cleared up after installation of seepage control measures in the canal and subsurface drainage would not be required.

There are few cases where the solution is that simple. Figure 4 shows a similar map that is more typical which shows the following:

1. The direction of ground-water flow is generally from north to south.
2. The average slope of the ground water tables is about 1 percent, which is rather steep, and indicates strong ground water flows in permeable soils.

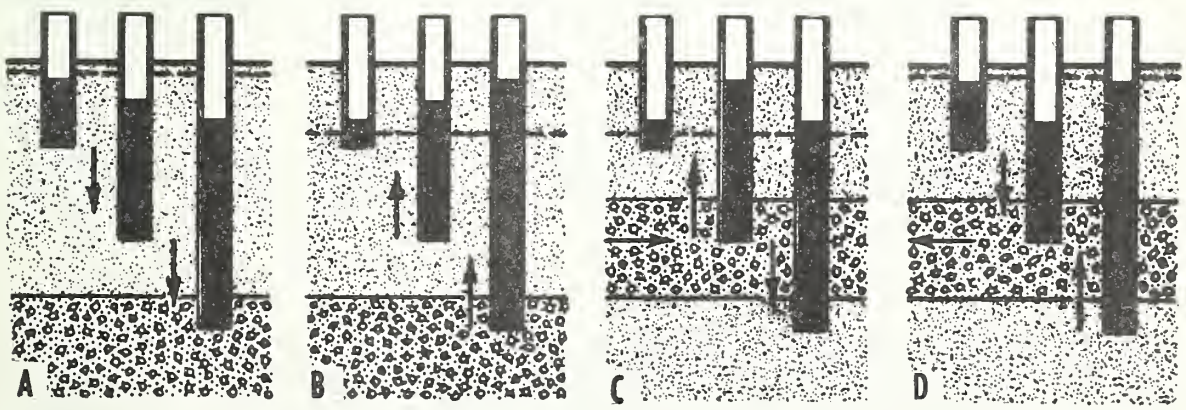

Figure 2.--The piezometers indicate: (A) That the ground water is going down and that there is some natural drainage; (B) a hydrostatic pressure or that there is water coming up from a deeper strata; (C) a hydrostatic pressure in a stratum, and that water is being forced both up and down from the stratum; and (D) that ground water is moving into a stratum and going out of the area.

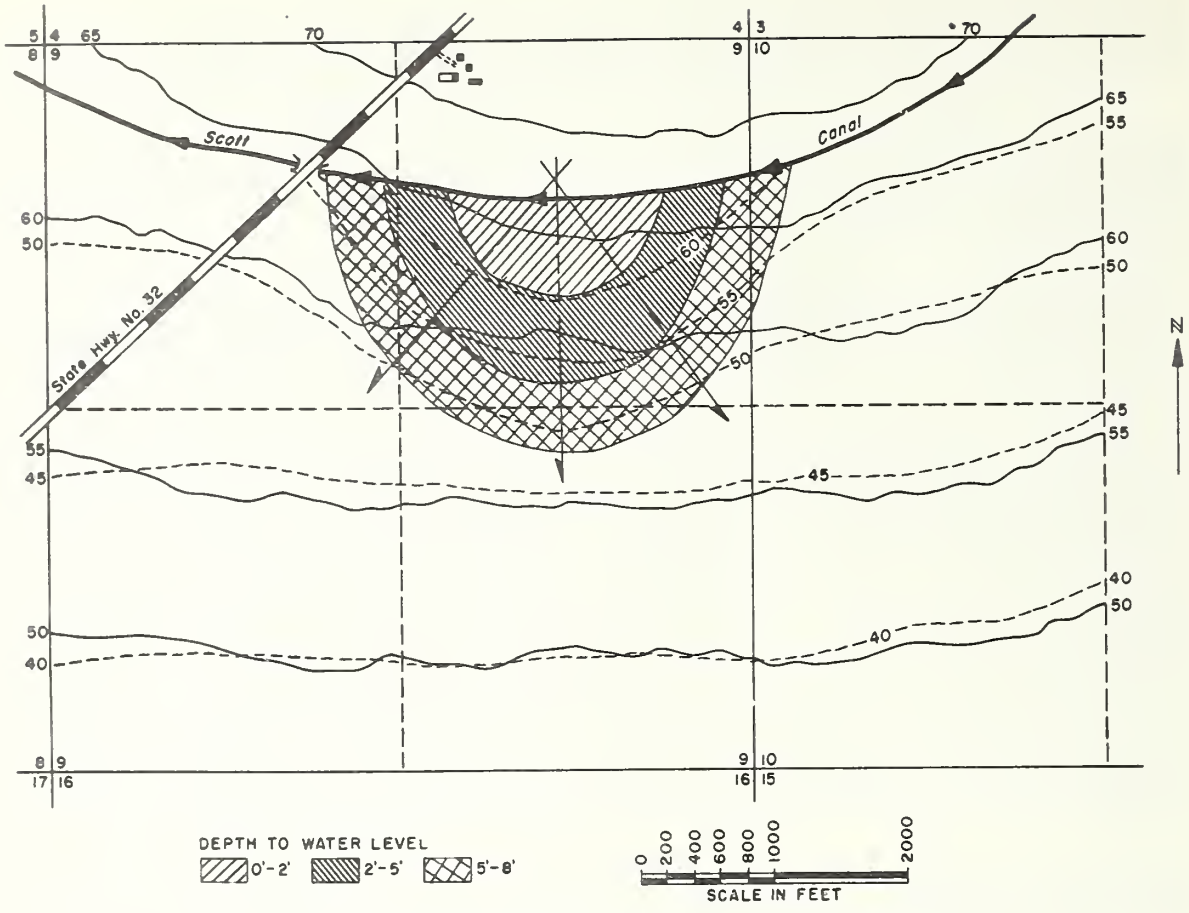

Figure 3.--Working map (canal seepage).

3. The location and extent of the land damaged by high water.
4. The configuration of the water table surface within the wet area.
5. The relative degree of wetness as indicated by the different cross-hatched areas.

In this case the source of the damaging water is most likely to be from a combination of the sources.

Again studies of canal losses and of quantities of underground flow would be in order to determine specific benefits of seepage control measures on the canals. Generally the correction of the land damage for areas of this kind requires installation of subsurface drains in addition to water management and seepage control measures determined to be feasible.

Most drainage investigations require the location and determination of at least the relative permeability of layers in the soil profile. In many cases damaging water moves through permeable layers of gravel or sandy materials approximately parallel to the ground surface. The constrictions in permeable layers, the presence of impermeable barriers underneath the ground surface or the changes in relative position of aquifers with respect to the ground surface cause many drainage problems. Some of these situations are described below.

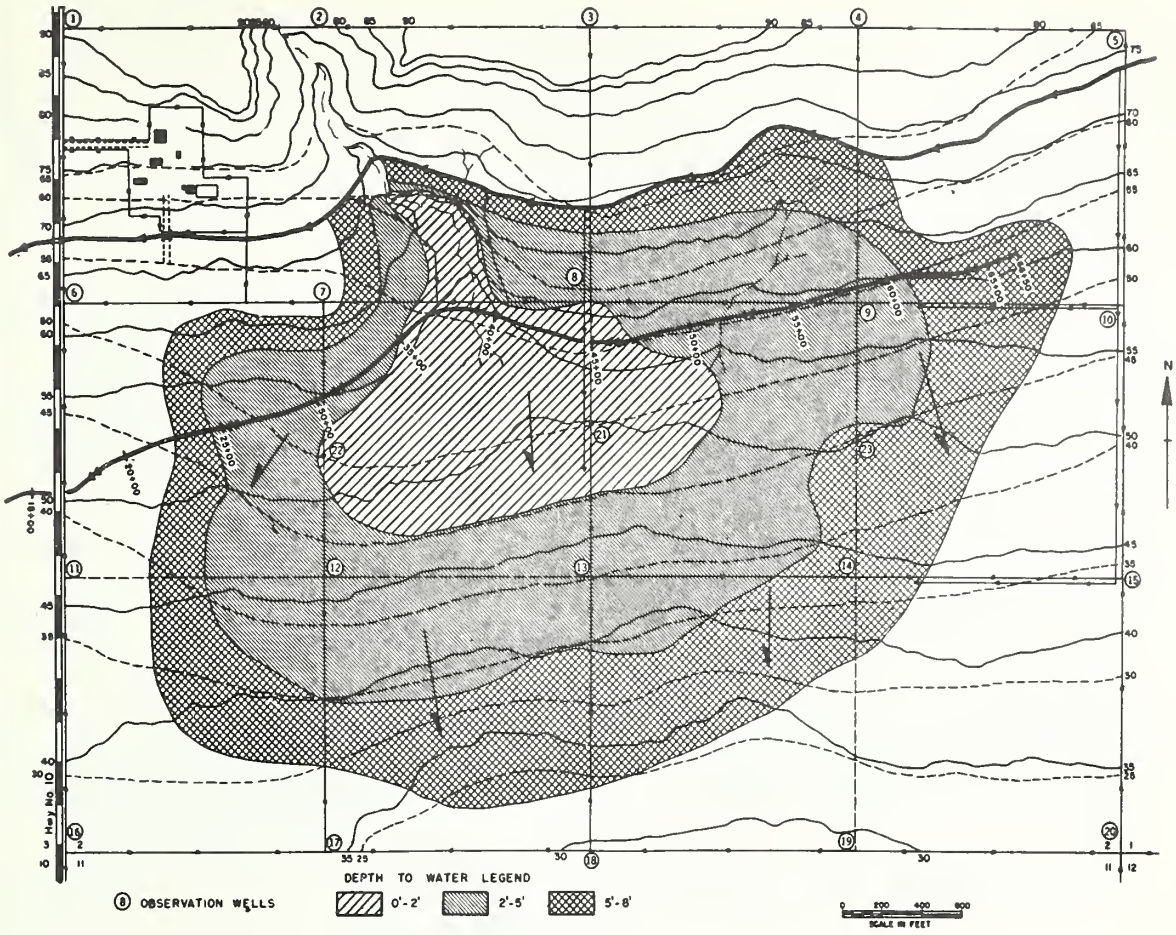

Figure 4.--Working map (completed).

Figure 5 shows a typical cross section of a valley area having a high water table. The solution to this problem requires the installation of an interception drain near the upper end of the seeped area on long slopes. In many cases it is necessary to locate two or more interceptors across the slope where the water rises to the surface.

Figure 6 shows a cross section where a seep area is caused by an impermeable barrier. The wet area may be cleared up by a drain above and parallel to the barrier.

Figure 7 shows a relief well discharging into a tile where permeable material is found below the drain.

These are some of the situations where ground water causes land damage and which may be cleared up by drains which are properly placed.

In all these cases seepage from canals, ditches, and reservoirs may be the source of part of the damaging water. Studies are needed to provide quantative estimates of canal losses and what effects seepage control measures would have on the ground water conditions. Where it is possible to establish the thickness of layer through which the ground water moves, it may be possible to estimate the underground flow by use of Darcy's Law.

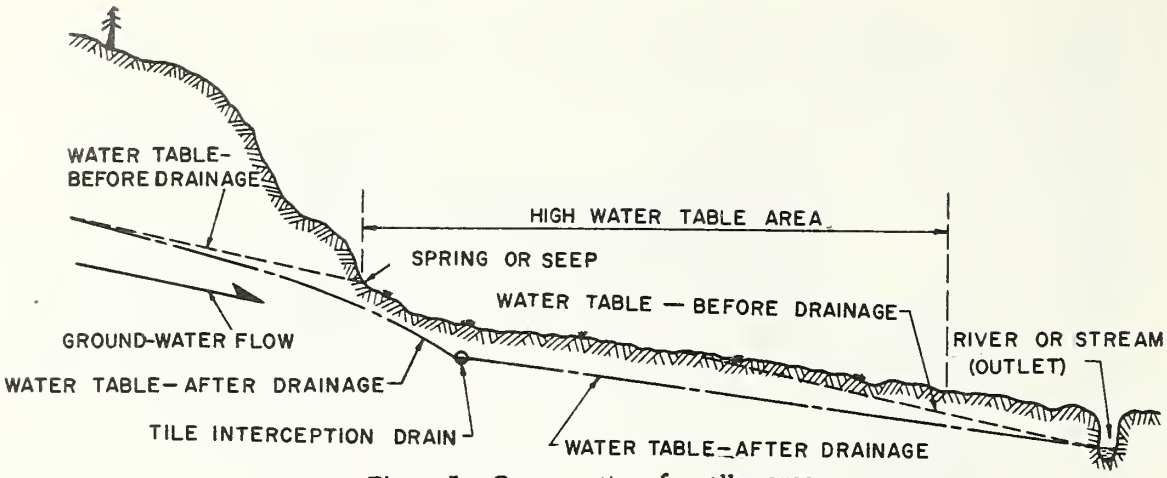

Figure 5.--Cross section of a valley area.

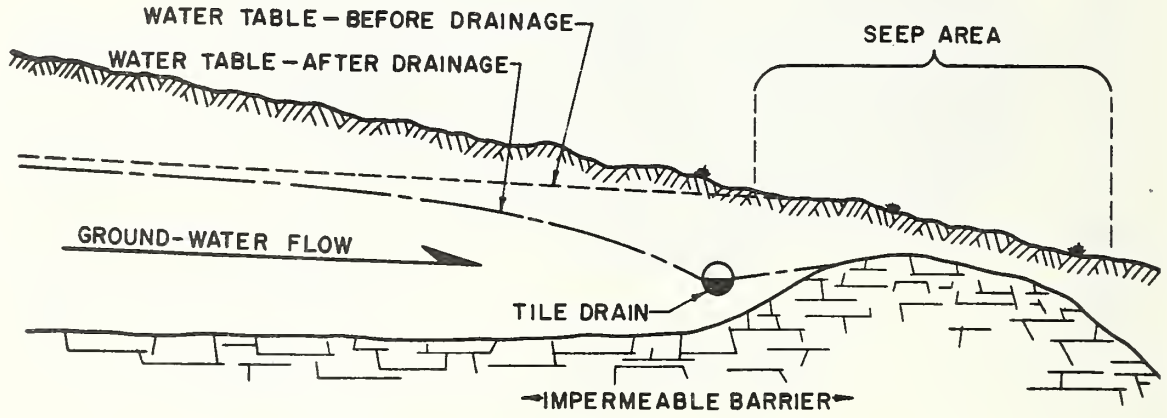

Figure 6.--Interception-type drain above an underground barrier.

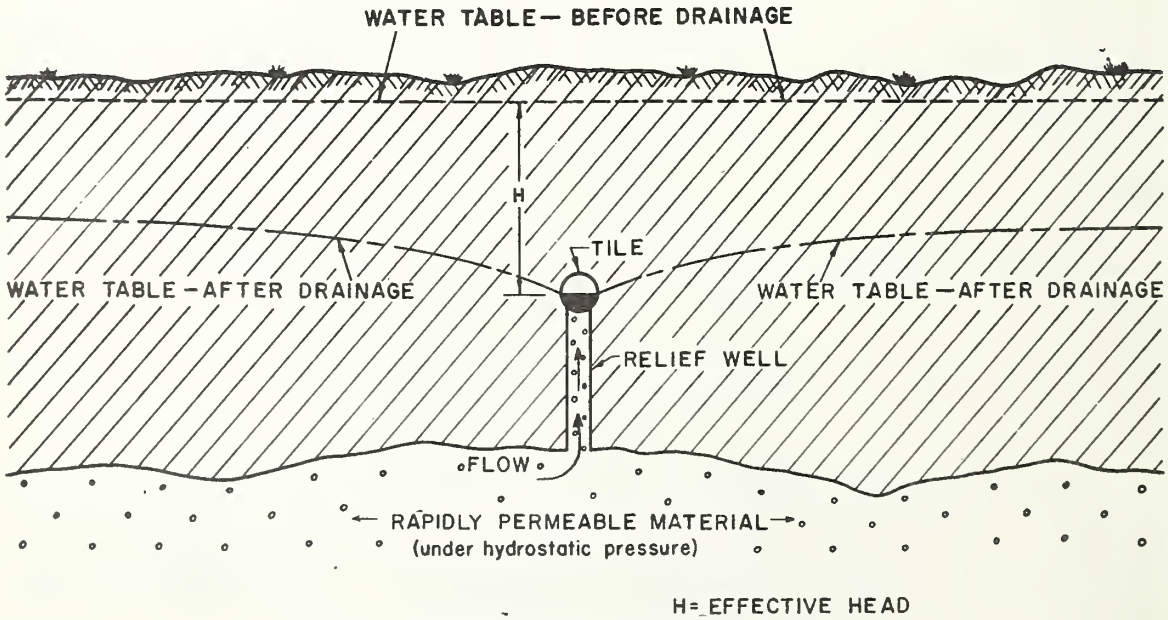

Figure 7.--Relief well drain discharging into tile drain.

The Darcy Formula $Q = PIA$ is basic to solution of ground water flow problems. In this formula Q is the discharge, P is the hydraulic conductivity or permeability, I is the slope of the hydraulic gradient and A is the cross sectional area of flow. Much progress has been made in research to provide aids in determining these factors. Methods of making investigations for wet irrigated land which are in current use by the Soil Conservation Service are described in a publication of the U.S. Department of Agriculture (4) and in current literature and I will only give a brief reference to these methods in this paper.

Hydraulic conductivity may be measured by the auger hole method (5) and by the double tube method (1) recently developed by the Agricultural Research Service.

The slope of the hydraulic gradient between two points may be determined by comparison of piezometer readings.

The thickness of the strata through which ground water moves may be determined by soil borings. Thus the quantity of ground water flow through a soil layer may be estimated by use of these methods.

Where the quantity of seepage can be determined, it may be compared with the total ground flow to estimate the effects of seepage reductions.

In the design of relief as well as interceptor drains, the theoretical spacing of drains may be reduced where the quantity of water to be handled by tile is decreased. This principle is illustrated by considering the effect of varying Q in the ellipse or Donnan equation which is often used in design of relief drains.

One form of this equation is:

$$S^2 = \frac{4K(m^2 + 2dm)}{P}$$

where

S is drain spacing in feet
K is hydraulic conductivity in inches per day
m is height of water table above drain at midpoint between drains
d is depth to an impervious layer below drain in feet
P is drain discharge in average depth over area drained by tile in inches per day (drainage coefficient)

This shows that the spacing of drains is proportional to the square root of the quantity of water to be handled. Actually an equivalent depth d_e should be used in this equation to obtain more accurate results (2) in use of this spacing formula, but this does not affect the general principle stated above.

Thus we may see that any improved water management or seepage control methods which are applied which reduce the quantity of water to be handled will reduce the intensity of drainage required.

Another factor which materially affects this subject is the reuse of drainage water. The quality of drain water varies greatly. In many cases drain water may be used directly or mixed with irrigation water. In other cases drain water is very high in salt and should be disposed of in a safe manner.

A brief summary of the topic damage to lands may be in order. Land damage in western irrigated areas is extensive and over one-fourth of the land irrigated has a problem of excess water. This is caused by ground water which comes from seepage, canals, reservoirs, and percolation beneath the root zone. The rise of saline and alkali salts generally affects the land damaged by a high water table.

The first step in arriving at recommendations for remedial measures is to make a thorough drainage investigation of site conditions. These investigations generally should include studies of the water table including heights at various seasons and hydrostatic pressures, subsoil borings to delineate depth and relative permeability of layers of soil through which the water moves, and often studies of quantity of flow and how such quantities may be reduced by seepage control measures. Also, the salinity conditions, soil, crops, water supply, etc., need to be considered with other site factors.

Seepage control measures, water management to improve efficiency of irrigation, and subsurface drainage are the principal methods of controlling the ground water table. Certainly the first two should be encouraged where practical to reduce land damage and conserve water. In a large proportion of cases of land damage due to high water table we encounter, it is necessary to install subsurface drains to obtain satisfactory control of the ground water and to control salinity. Where it is possible to reduce seepage through improved farm efficiency including land leveling, etc., and through seepage control measures, the quantitative effect on ground water flow should be considered. Where it is possible to install these measures, it may be feasible to reduce the intensity and cost of drainage; or if they are installed after drainage, then the drains will be more effective since they will need to handle less water.

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SEEPAGE PROBLEMS IN WEST PAKISTAN[1]

Charles H. Welker

With the announcement of the Symposium on Seepage and the themes for discussion, we at Harza Engineering Company were immediately interested in hearing the presentations. We are indirectly involved in one of the largest reclamation projects in the world in which a major contributing factor has been the seepage loss from unlined canals.

This project is in West Pakistan in south Asia (fig. 1). Pakistan was formed as a separate nation from the predominantly Moslem parts of colonial India when Britain gave these countries their freedom in 1947. The map shows its position with respect to India and its other neighbors. In population, Pakistan's 90 millions make it the sixth largest in the world.

WEST PAKISTAN

West Pakistan, with an area about one-third that of the contiguous 48 States, has a population of about 40 million. Climatically it is somewhat like Arizona; most of it is desertlike with a low annual rainfall.

Temperatures vary between wide limits. Extreme highs occur in the summer, often reaching 120 degrees and remaining high for long periods. Frosts often occur in winter, although the Himalayas shield the area from most of the frigid blasts. Annual rainfall ranges from less than 5 inches in the southern areas to 30 inches in the Himalayan foothills, averaging less than 10 inches for most of the area. This 750-mile-long Indus

Figure 1

[1] Contribution from Harza Engineering Company, Chicago, Illinois.

Valley area from the foothills to the Arabian Sea is habitable only because of the life-giving waters furnished by the Indus and its tributaries.

INDUS RIVER BASIN

The Indus, starting in the Tibetan Plateau north of the Himalayas carries a substantial year-round flow comprised of snow melt as well as monsoon runoff. Five major tributaries contribute additional large flows, but mostly from monsoon runoff.

All five of these tributaries and the Indus itself have their origins in India or Indian-controlled territory. They spill out of the hills onto unbelievably flat plains which have no significant differentials in elevation and a gentle land slope of only about 1 foot per mile toward the Arabian Sea. This vast area is actually a deltaic plain. The Indus and its five tributaries flow out across this plain in wide shallow meander depressions joining each other until they form one river, the Indus, which flows about 450 miles farther until it enters the sea (fig. 2).

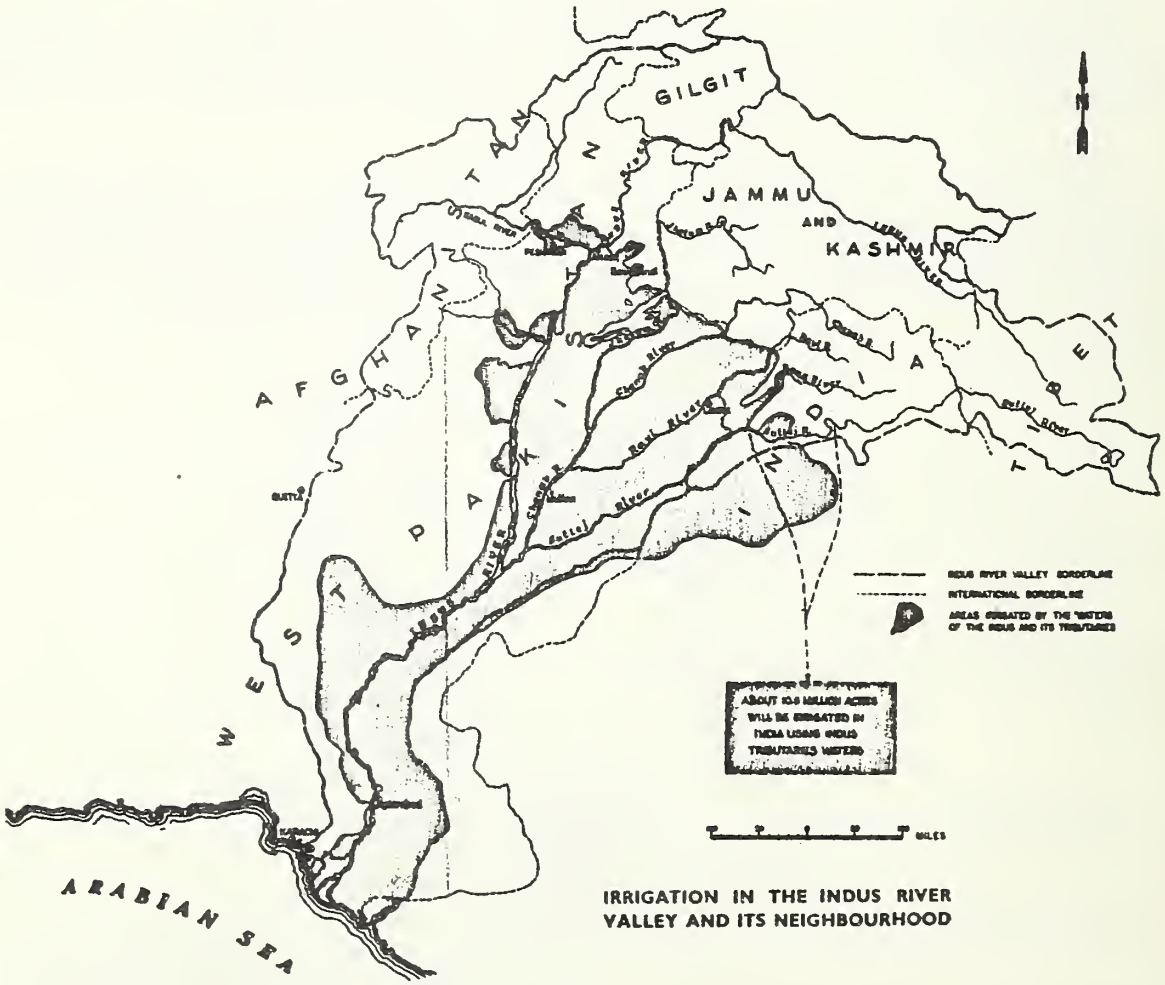


Figure 2

Geologists say the Indian Peninsula was a matured stable block when the Himalayas and its foothill ranges were formed. Between them was a wide deep depression which was gradually filled with alluvium by the Indus and Ganges river systems flowing from the mountain areas. This plains area is known as the Indo-Gangetic Plain. The deposits are composed of unconsolidated fine sands and silts filling depths as great as 20,000 feet. Buried mountain ranges raise their peaks above the surface at a few places, but generally the deposits are so deep that their full depths have not been probed to bedrock.

The Indus, Ganges, and other rivers gradually filled the trough with their delta deposits to its present levels, often shifting their courses, working and reworking the deposits, laying down discontinuous layers of sand, silt, and clay. There is evidence the Ganges river once flowed parallel to the Indus to the Arabian Sea, but later shifted and developed its outlet with the Brahmaputra in the Bay of Bengal on the eastern side of the Indian Peninsula.

DEVELOPMENT OF IRRIGATION

Man learned to be a farmer in the Indus Valley about the same time his cousins learned farming in the Tigris-Euphrates Valley and the Nile Valley. The conditions were somewhat similar; a suitable climate, fertile soil, and a river that assured a year-round water supply, whose floods thoroughly wetted the valley floor enough to nurture a crop planted as soon as the floods subsided. Ruins of two major cities have been found which indicate that a high degree of civilization flourished in the region 5,000 years ago, then died again for unknown reasons.

Inundation canals were developed in the area over 200 years ago, feeding flood waters into areas well back from the rivers. Under the pressure of expanding population, the British undertook development of more extensive canal systems, and in 1882 completed the first diversion dam, in this case a weir, thereby enabling them to turn water into the canal regardless of the river stage.

Improvements and extensions of existing inundation canals and additional canal development projects gradually increased the irrigation systems until today 23 million acres are under canal irrigation in West Pakistan, virtually all supplied by the Indus River and its tributaries. This comprises about 10 percent of the total irrigated acreage in the world and is the largest irrigated area supplied by a single river system. Its area is about equal to the total acreage irrigated by rivers in the contiguous States of America. The Nile Valley has only about 7 million acres under irrigation, whereas one barrage on the Indus, that at Sukkur, has this much acreage under command.

There are now 15 barrages and weirs in Pakistan diverting waters into more than 30 thousand miles of canals. The present water uses total about 72 million acre-feet, almost the total dependable flow of the presently unregulated flow in the rivers. Annual runoff of the Indus ranges from 160 to 240 million acre-feet, with an average of about 168 million acre-feet. Under the Treaty signed with India in 1960, 33 million acre-feet will be diverted on the Indian side of the border to irrigate about 10 million acres. Within Pakistan, two large storage reservoirs plus 350 miles of large canals are to be built to replace the irrigation supplies in the eastern rivers which will be cut off by the Indian diversions.

RISE OF GROUND WATER LEVELS

Less than 5 percent of the canals in Pakistan are lined. Those which are lined have relatively high seepage losses, as linings are generally made of a double layer of locally made bricks laid in mortar. Seepage losses from the canals and rivers are rather high, as one can visualize that canals having a perimeter of river-deposited sands and silts would be rather pervious. Losses are estimated to be 35 to 40 percent of the volumes diverted from the river. Part of this is evaporation, especially in the high temperatures of the summer, but most of it is lost to the ground water.

In over half of the irrigated area, the small annual rainfall, ranging from 2 to 10 inches, is too light to contribute to the groundwater. However, the river floods in the monsoon season, with frequent overbank flooding, contribute substantial volumes to the ground water. With the recession of the floods, much of the water which soaked into the banks seeps back into the river channels, thereby helping to sustain the river flows during the remaining months of the year. Plottings of the ground water contours of about 60 years ago, prior to the advent of the present large-scale irrigation, indicate the ground water surface sloped from the rivers toward the centers of the land areas (doabs) lying between them. There was a general ground water slope in the center of the doab toward the Arabian Sea. Profiles across the doab show that the land surface is only 10 to 20 feet above the normal river levels, but the ground water level at the center of the doab in 1900 was as much as 70 feet below the ground level.

During the years since then, the ground water levels have risen steadily because of the additions from canal seepage until today the ground water level is within a few feet of the surface (fig. 3). As the level approached the ground surface, the evaporation and transpiration increased, tending to reduce the rate of rise.

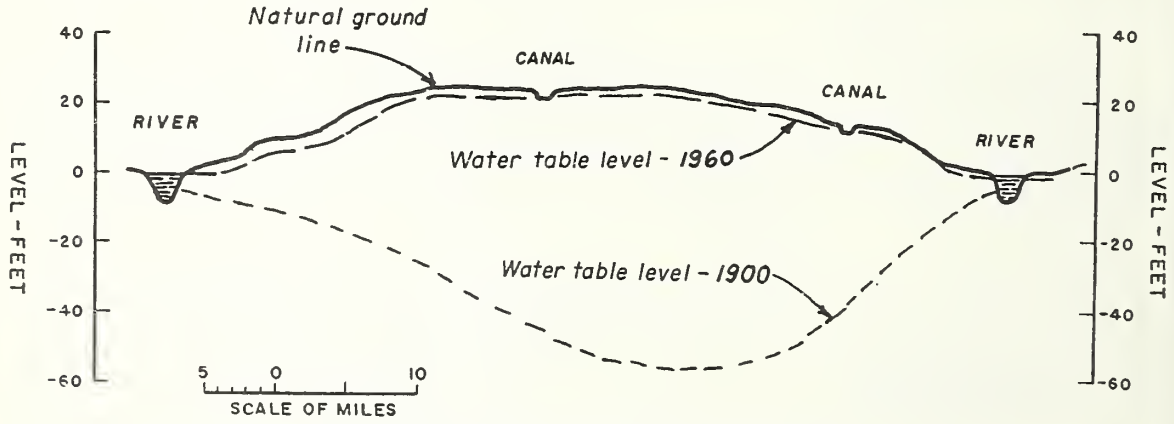


Figure 3.--Typical section across a doab.

INCIDENCE OF SALINITY

Salinity of the soil, with its attendant damage to crops, has been gradually developing. The waters from the Indus and its tributaries have a relatively low dissolved solids content, having only about 200 to 300 p.p.m., and as such are considered high quality waters for irrigation. Unfortunately, these waters have been used too sparingly. Diversion of 72 million acre-feet less losses of 27 million acre-feet leaves only 45 million acre-feet to irrigate 23 million acres. This with annual rainfall of less than 1 foot gives the land less

than half what it needs for maximum crop development. Therefore little if any of the applied water infiltrates to the groundwater, and the residual salts accumulate in the root zone.

RECLAMATION PROPOSALS

The concurrent rise of ground water levels, causing waterlogging, has complicated the reclamation program. Leaching the salts from the root zone by proper use of the irrigation water would be relatively easy if there were room in the ground for the leaching water; but with the high ground water levels, application of leaching waters is not feasible. In fact, the high ground water levels aggravate the problem, because capillary action brings water to the surface where it evaporates, leaving its residue of salts to accumulate on the ground surface.

Drainage ditches and tile lines normally used on other reclamation projects generally cannot be used in Pakistan to relieve the waterlogging problem because there is too little topographic relief to provide suitable gravity outlets. This drainage method is feasible only in a few locations where dictated by special conditions. Deep gravel-packed wells with high-capacity turbine pumps generally provide the cheapest and easiest means for removing the ground water to lower the water table. In the northern zones near the foothills, the ground water is of relatively good quality. In some cases it is "sweet" enough to be used directly for irrigation and at other places can be mixed with surface supplies in the canals which can dilute it to satisfactory use standards. High-capacity wells of 3 to 8 cusecs can be constructed having high specific capacities, thereby reducing the number of wells to be constructed without the handicap of operating them with excessive pumping heads. Since the present canal supplies are deficient in satisfying the full crop needs, much or all of the water, where it is of satisfactory quality, can be utilized on lands near the wells by using it for supplemental irrigation.

GROUND WATER QUALITY

The canal seepage losses which contributed to the rise of the water table were generally good quality waters. But the native ground waters resulting from infiltration of river channel waters and overbank flood waters were not always of such good quality, possibly because they had higher salt content from their leaching action of cumulative salts left by earlier floods. At any rate, what might first appear to be a large saturated aquifer of almost limitless quantities of useable water is not necessarily that. Exploratory work is now in progress in much of the reclamation area to determine the ground water quality and extent of the usable waters, which will, in turn, establish what means can be used to lower the water table, what uses can be made of the water, or what means should be provided for its disposal.

Where "sweet" water from canal seepage forms a lens of limited depth, large-capacity wells cannot be used for recovery of the sweet water, as the well would draw the saline water lying below the good water. Instead, numerous small-capacity wells may be needed. If such means cannot recover substantial quantities of good water for supplemental use, it may be better not to pump the saline water out. Its disposal to a distant river may be costly, and disposal into the river may impair the quality of the river water for use at some other diversion point farther downstream. In such circumstances, only ditches and tile lines can be used for lowering the ground water table. Some reclamation projects along the lower Indus have ground waters whose salinity is considerably greater than sea

water. For these projects, surface drains provide the only permissible means for lowering the ground water levels.

CURRENT RECLAMATION PROJECTS

Some reclamation efforts have been attempted in the northern zones. Most of these had only a limited degree of success, because the project areas were too small. Ground water inflow from peripheral areas or from nearby canals made the task of lowering the ground water level too difficult. A large reclamation project, comprising an area of about 1 million acres is about to be put in operation just west of Lahore. Some tube wells, constructed for earlier reclamation attempts, were already in the area. An additional 1,800 wells have been drilled and are almost ready to begin work. Electrification of the area was concurrently necessary to provide power for the pumps, since very little electric power and few transmission lines were available in the area when the project was started 3 years ago. This large project unit will permit verification of the hydrologic conditions assumed in its design. An evolution of revised agricultural and irrigation practices will also be necessary in order to utilize the reclaimed ground water and gain the full benefits which might accrue. Other major project units are being planned, with the goal of ultimate reclamation of all areas now under irrigation.

CANAL SEEPAGE

There is no certainty about the rate of seepage from the canals to the ground water. Pakistan irrigation engineers have estimated their seepage loss rate from unlined canals to be about 8 c.f.s. per million square feet of wetted perimeter, or 0.7 cu. ft. per sq. ft. per 24 hrs. For a canal carrying 10,000 c.f.s., seepage losses would be about 12 c.f.s. per mile. Efforts to check this seepage loss rate by precise current meter traverses at each end of long reaches of canal have been attempted. Careful measurements have indicated these seepage loss rates may be as much as 35 c.f.s. per million square feet, or four times as high as the normal rate. If the seepage rates are excessive, reclamation will be difficult and costly.

CANAL LININGS

Lining of operating canals was not economically feasible in the early days of the irrigation development. Water was sufficiently abundant and the cost of the canal systems so low that the additional revenues which would have been realized from the lost seepage water could not, either then or now, justify the cost of the linings to reduce the seepage loss. Only now, when the twin specters of waterlogging and salinity menace the lands and cause a great loss of agricultural output to the detriment of Pakistan's economy, does the installation of linings become justified.

However, the ground water levels are now too high to permit ordinary linings to be installed without potential danger of lining failure whenever the canal water levels drop too rapidly. Also, most main canals operate almost continuously, seldom being shut down for more than 2 or 3 weeks per year. Therefore, shutdowns for installation of linings would not be permissible. Some engineers have differing opinions about the handicap or value of seepage loss from the canals. Some well-qualified men infer that if an inexpensive means of sealing the canals were devised and utilized to stop the present seepage losses, thereby making it easier to gain control of the water table, there may be a time when these

linings should again be removed. The stated purpose is that increased seepage would simplify the recharge of the groundwater from the canals in order to permit continued "mining" of it for supplemental irrigation. However, logic indicates that if recharge of the ground water is necessary, it could be done as well and with greater benefit by using an equivalent volume of water for reclamation of saline lands, thereby performing a useful operation with the water before it is added to the ground water reservoir.

RECLAMATION AND RELATED PROBLEMS

Regardless of the means employed, Pakistan is faced with a monumental task of reclaiming several million acres of lands already stricken to some degree by salinity, with the problem complicated by high ground water levels caused by seepage losses from her unlined canals. The problem is doubly urgent, because she is already unable to feed her millions with the food she now raises. The reclamation effort will require the solution of a host of special problems if it is to succeed. The effort would tax the resources and skills of the USA with all its technology. It will require that much more effort from a country which is not so abundantly blessed as ours.

I have already touched lightly upon the problem of contamination of downstream irrigated areas by release of highly saline waters pumped from contaminated ground water sources in attempting to lower the water table. Leaching of saline soils will contaminate the upper layers of ground water, making it less desirable for reuse. Increased use of fertilizers will add to the mineral content of the ground water when some of it is also leached through the root zone.

The construction and operation of the wells or drains and the parallel development of electrification to operate pumps requires a high degree of mechanical skills from people who have few such native skills and also have a low rate of literacy. Tremendous financial resources are necessary. But full benefit and utilization will also require an upgrading of farming skills, development of improved farming methods, use of improved seeds, revisions in crop patterns, and solution of a multitude of corollary problems.

PRESENT RECLAMATION ACTIVITY

One American consulting firm, Tipton and Kalmbach, Inc., of Denver, along with other consultants, is already engaged in design and construction of reclamation project units in Pakistan in an effort to combat the salinity and waterlogging problems. American contractors like H. T. Smith, Inc., are drilling some of the wells. The work is being financed by the World Bank, U.S. aid funds, and gifts or long term loans from other sources.

Pakistan's President Ayub Khan appealed to President Kennedy about 18 months ago for technical help. Kennedy sent a commission of scientists and engineers to Pakistan in the fall of 1961 to survey the situation and report on a program of how the work should be carried out. Their report is under preparation and has not yet been released. Some men here today participated in that effort.

The reclamation work is being carried out under the direction of the Water and Power Development Authority of West Pakistan (WAPDA). This governmental corporation is also entrusted with the planning and construction of a multitude of civil works projects including expansion and extension of the electric power generating and transmission facilities, construction of the dams and canals required to implement the Water Treaty signed with India in 1960, and a number of other projects for irrigation development.

Harza is involved in these as general consultant to the Water and Power Development Authority. As general consultants, we have assisted them by reviewing and coordinating project planning for most of the projects in which they are engaged, as well as advising them on establishing and directing agencies to gather basic data for future projects, and furnishing personnel for specialized assistance where needed. We have also been preparing a report appraising their natural resources of water and power. This report will be utilized in coordinating the planning of works which will enable the people of Pakistan to achieve, as rapidly as possible, full economic independence as well as political independence.

Some of you gentlemen have already contributed toward solution of the problems in Pakistan. Others of you may be called upon to contribute your skills and knowledge. All of you will recognize that Pakistan's reclamation problem is as old as man's earliest efforts to develop irrigation agriculture. It is especially significant today because of its size and the urgency with which the problem needs to be solved.

ECONOMIC COMPARISONS OF CANAL LININGS FOR THE CALIFORNIA AQUEDUCT[1]

J. A. Wineland and C. V. Lucas[2]

INTRODUCTION

The purpose of the study reported herein is to compare the economics of various canal linings as a guide in selecting a type of lining which will be further studied in detail before its adoption for use in the California Aqueduct.

The California Aqueduct will extend from the Sacramento--San Joaquin River Delta near Tracy southward to terminate in Riverside and Los Angeles Counties. The California Aqueduct is located along the west side of the San Joaquin Valley from the Delta to the Tehachapi Mountains. South of the Tehachapi Mountains it bifurcates into two branches; the East Branch which terminates at Perris Reservoir near Riverside, and the West Branch which terminates at Castaic Reservoir just north of Los Angeles. The length is approximately 450 miles. The aqueduct varies in capacity from a maximum of 13,100 second-feet in the San Luis Division down to 2,800 second-feet at the southern end of the San Joaquin Valley and to 730 second-feet at Perris Reservoir.

The portion of the California Aqueduct considered in this study includes four reaches within the San Luis Division with capacities of 13,100, 11,800, 9,350, and 8,350 c.f.s. The facilities in this division are to be jointly used by the U.S. Bureau of Reclamation and the State of California. The aqueduct facilities in this division are being designed and constructed by the Bureau of Reclamation and are to be operated by the State of California, Department of Water Resources, as an integrated portion of the California Aqueduct system. Approximately 55 percent of the capacity of these facilities will be used to serve the State of California project needs while 45 percent of the capacity will be used to serve the Federal San Luis project needs.

[1] Contribution from the Resources Agency of California, Department of Water Resources, Sacramento, Calif.

[2] Chief and head, respectively, of the Soil Design Unit, Aqueduct Design Branch, State Department of Water Resources.

GENERAL CONSIDERATIONS

This study was made by the Department of Water Resources to compare earth- and concrete-lined canals, various thicknesses of concrete linings and several special sublinings.

The factors considered to be of major importance and included in this study are:

- Cost to replace water lost to seepage and evaporation.
- Operation and maintenance costs.
- Initial construction costs.

Factors which are to be the subject for further study are:

- The effect of lining thickness on construction costs.
- The potential life of special sublinings.
- The additional construction problems with sublinings.

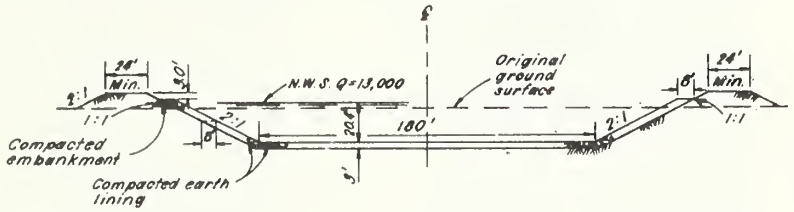
However, these factors were considered to be significant and unit construction costs which probably would be affected were increased by 20 percent.

Factors such as escalation of construction costs and additional right-of-way costs were considered but were of small dollar value and for purposes of this analysis are not included.

We have compared the "Standard" canal linings for large canals and several special linings. The "Standard" linings are heavy compacted earth and unreinforced concrete. The special linings are composed of a 4-inch concrete lining with a film of catalytically blown asphalt or vinyl plastic as a sublining. In the past, special linings to control seepage from canals have not been used extensively because they were too expensive to construct or the water saved by lining was not of sufficient value. Another factor was that the water lost due to seepage could in some cases be recovered within the project area. With the increasing cost of constructing conservation and transportation facilities, the water is becoming more valuable and new or different facilities to reduce losses can be justified.

ASSUMPTIONS AND CRITERIA

The length of the San Luis Division Canal is approximately 86.4 miles and for purposes of this comparison was divided into four sections. The typical sections and hydraulic properties of the four sections are shown on figures Nos. 1, 2 and 3. The heavy compacted earth-lined sections utilize selected earth materials to form a seepage barrier approximately 3 feet thick. The typical concrete-lined sections shown in figure 2 would have the same physical dimensions for both the 4- and 5-inch lining thicknesses considered. Notice the difference in wetted perimeter and water surface for the heavy compacted earth sections and concrete sections.



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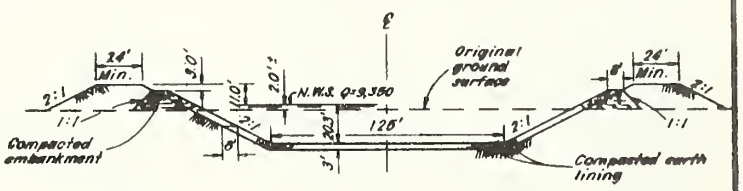

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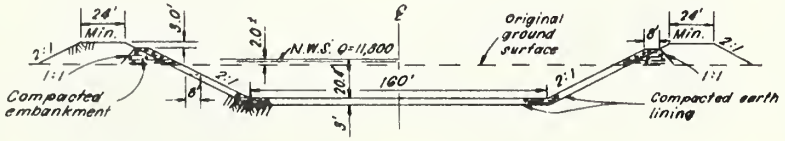

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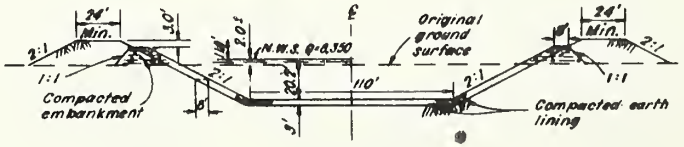

SECTION NO. 4

HYDRAULIC PROPERTIES

SECTION NO.	A	V	Q	r	n	S
1	4504	2.91	13,100	16.61	.021	.00004
2	4096	2.88	11,800	16.30	.021	.00004
3	3382	2.79	9,350	15.58	.021	.00004
4	3038	2.74	8,350	15.18	.021	.00004

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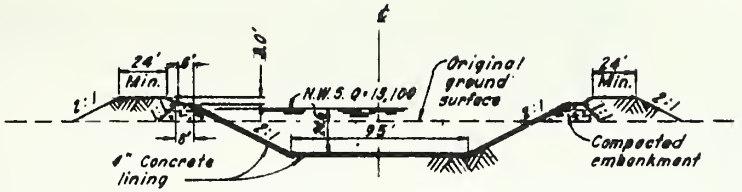
DEPARTMENT OF WATER RESOURCES
DIVISION OF DESIGN AND CONSTRUCTION

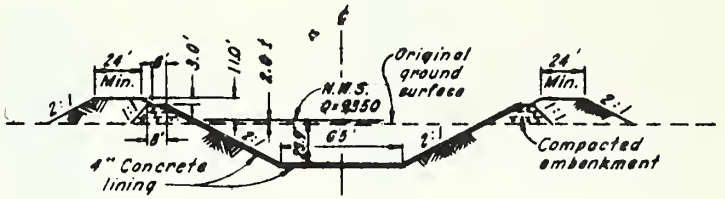
CALIFORNIA AQUEDUCT
SAN LUIS DIVISION

TYPICAL EARTH LINED SECTIONS

Figure 1



SECTION NO. 1



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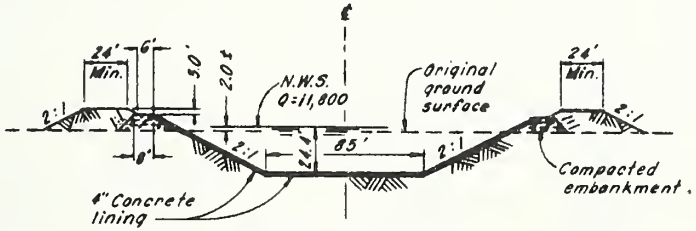

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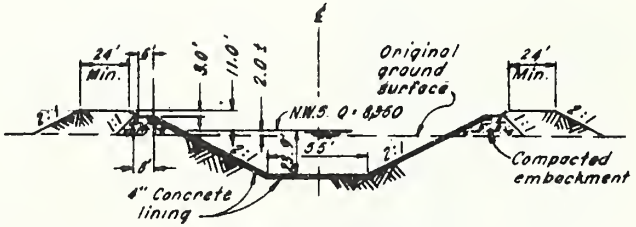

SECTION NO. 4

HYDRAULIC PROPERTIES

SECTION NO.	A	V	Q	r	n	S
1	3547	3.70	13,100	17.30	0.17	.00004
2	3265	3.63	11,800	16.82	0.17	.00004
3	2696	3.46	9,350	15.68	0.17	.00004
4	2457	3.39	8,350	15.18	0.17	.00004

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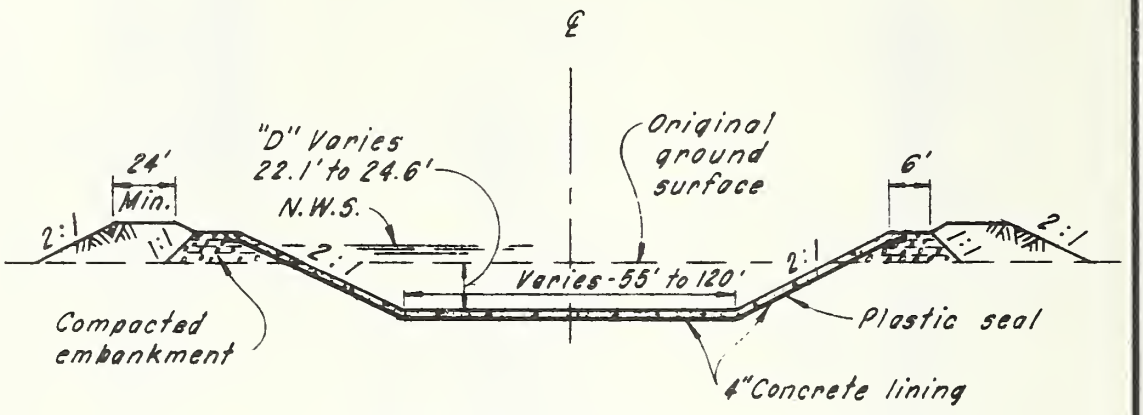
DIVISION OF DESIGN AND CONSTRUCTION

CALIFORNIA AQUEDUCT
SAN LUIS DIVISION
TYPICAL CONCRETE LINED SECTIONS

Figure 2



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DIVISION OF DESIGN AND CONSTRUCTION

CALIFORNIA AQUEDUCT
SAN LUIS DIVISION
4" CONCRETE LINING
WITH SUBLINING

Figure 3

SEEPAGE RATES

Seepage rates for various linings are as follows:

Lining type	Assumed seepage rate, cubic-foot per square foot of wetted surface per day
Unlined canal	0.50
Heavy compacted earth	[1] .07
5-inch and 4-inch concrete	[1] .07
4-inch concrete with asphalt sublining (75 percent of concrete rate)	.0525
4-inch concrete with plastic sublining (50 percent of concrete rate)	.035
4-inch concrete with plastic sublining (5 percent of concrete rate)	.0035

[1] This seepage rate is an average figure for other large canals in the vicinity. Refer to " U.S. Bur. of Reclamation Lower Cost Canal Lining Program" Amer. Soc. Civil Engin. Jour. Irrig. and Drainage 84 (1R2). 1958

The seepage rate assumed for the 4-inch concrete lining with the asphalt membrane sublining was 75 percent of that for the concrete lining or 0.0525. The seepage rate assumed for the plastic sublining was 50 percent and 5 percent of the concrete lining or 0.035 and 0.0035, respectively.

There is a paucity of data on seepage through concrete on sublinings, and because of this circumstance the above wide range of assumed values was used in order to cover fully and evaluate conditions that future data may substantiate. Preliminary data on seepage through concrete on asphalt and plastic in our South Bay Aqueduct indicate that these sublinings may do an excellent job of seepage control. It has been observed that in areas with a "Standard" concrete lining the subgrade is nearly saturated while in areas with sublinings under the concrete the subgrade has had only a slight increase in moisture content.

EVAPORATION RATES

The evaporation rate used was 8 feet per year. This rate varies throughout the San Joaquin Valley and could be as low as 4 feet and as high as 12 feet, depending on the location.

ECONOMIC CONSIDERATION

A project life of 50 years and an interest rate of 3 1/2 percent was used.

OPERATION AND MAINTENANCE COSTS

Cost of Pumping Water

The total for average pumping costs for replacement water delivered to the San Luis Division was estimated at $1.00 per acre-foot.

The operation and maintenance costs given below are the additional costs for an earth-lined canal over and above those for a concrete-lined canal.

Additional Operating Costs for Earth-Lined Canal

Turnouts.--Additional cost for installing turnout structures after the canal has been placed into operation. (This is principally due to greater cost in cofferdaming the site.) Eleventh through 30th year of operation - $5,000 per year.

Additional Maintenance Costs for Earth-Lined Canal

(For entire reach of canal)

	Additional annual cost
1. Lining repairs---------- (It is estimated 5 miles will be rebuilt in the first 10 years.)	$25,000
2. Canal cleaning----------	60,000
3. Embankment maintenance ----------	91,400
a. Replacement of materials in beach belt	
b. Replacement of riprap	
c. Repairs to embankment	
4. Weed control----------	47,500
5. Structure maintenance - longer transitions for earth-lined canal----------	10,000
6. Pump and meter maintenance----------	45,000
Additional maintenance cost total ----------	$278,900

VALUE OF WATER

In the San Luis Division, a portion of the water transported is for Federal project use and a portion for State project use. For purposes of this report, 55 percent of the flow is assigned to the State project and 45 percent is assigned to the Federal project. The Federal project seepage loss values are equivalent to costs of recovering the seepage water by pumping from ground water within the Federal San Luis service area.[2]

COMPARISON OF CANAL LININGS

The first comparison is made to determine whether a lining of any type would be economically justified. In table 1, an unlined canal is compared to a canal with heavy compacted earth and a 4-inch concrete lining. Operational and maintenance costs were not included in this comparison because it was an unnecessary refinement and would not change the results other than to further justify lining. The unlined canal was assumed to have the same physical dimensions as the heavy compacted earth-lined canal shown in figure 1.

The second comparison is between a concrete-lined canal and a canal with heavy compacted earth lining (table 1). The heavy compacted earth lining is used as the basic section, and the cost of the incremental water recovered or saved by using concrete is considered.

The third comparison uses 4-inch concrete as the basic section and compares it to 4-inch concrete with various sublinings. (table 1)

As shown in table 1, cost of the incremental water recovered by using a specific lining type is compared to the cost of replacing this incremental water from an alternate source. The cost of water recovered by using a specific lining type is based upon the incremental construction costs, incremental operations and maintenance costs, and the incremental water recovered. The cost of water from an alternate source is the combined value (Federal and State) of the incremental seepage and evaporation losses.

CONCLUSIONS

A lining for this canal is justified. When comparing concrete linings with heavy compacted earth linings a 5-inch concrete lining cannot be economically justified. Considering the accuracy of the analysis, a 4-inch concrete lining may be used in lieu of a heavy compacted earth lining.

A 4-inch concrete lining with sublinings is justified if other problems such as those concerned with construction are proven by further study to be practicable. Other sublinings than those used in this study may be found more suitable.

Not included in this report, but of general interest, is the determining of the minimum economic life of a plastic sublining. Using the criteria in this report for the plastic sublining with 95 percent seepage reduction, a minimum life expectancy of 12 years would make its use economically feasible.

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[2] U.S. Bureau of Reclamation unpublished report, "Preliminary Design of Canal Sections San Luis Canal" Rev. September 1961.

TABLE 1.--COMPARISON OF STANDARD CANAL LININGS TO UNLINED CANALS

Lining type	Construction cost	Incremental construction cost	Annual water losses		Incremental annual water recovered by lining type	Present worth, incremental water recovered	Present worth, incremental operation and maintenance cost	Cost of incremental water recovered per acre-foot	Cost of water alternate source, per acre-foot
			Seepage	Evaporation					
			Acre-feet	Acre-feet	Acre-feet				
Unlined.....	$22,300,000	----------	459,000	24,000					
Heavy compacted earth..	30,700,000	$8,400,000	65,000	24,000	394,000	$9,240,000		$0.91	$13.10
Four-inch concrete.....	46,100,000	23,800,000	53,000	14,000	416,000	9,759,000		2.44	13.25
Comparison of Concrete Linings to Heavy Compacted Earth Lining									
Heavy compacted earth..	30,700,000		65,000	24,000					
Five-inch concrete.....	52,200,000	21,500,000	53,000	14,000	22,000	516,000	-$7,100,000	27.91	16.00
Four-inch concrete.....	46,100,000	15,400,000	53,000	14,000	22,000	516,000	- 7,100,000	16.08	16.00
Comparison of Concrete With Sublinings and "Standard" Concrete Lining									
Four-inch concrete	46,100,000		53,000	14,000					
Four-inch concrete, asphalt sublining....	49,300,000	3,200,000	39,000	14,000	14,000	328,000	- 328,000	8.76	13.10
Four-inch concrete with plastic sublining, 50 percent seepage reduction.....	52,700,000	6,600,000	26,000	14,000	27,000	633,000	- 633,000	9.43	13.10
Four-inch concrete with plastic sublining, 95 percent seepage reduction	52,700,000	6,600,000	3,000	14,000	50,000	1,173,000	- 1,173,000	4.63	13.10

DISCUSSION

Tuesday morning - Second Session

Pugh:

I have a question for Mr. Lucas. He kept mentioning an alternate source of water in your economic comparison. What was the alternate source in this California canal?

Lucas:

It was the development of other projects within the Sacramento Valley complex, primarily, the Oroville area, and the Eel River area.

Pugh:

Isn't this California canal a part of the Feather River Project?

Lucas:

Yes, the water from the Feather River development is delivered into our San Joaquin River and Sacramento River delta, and it is pumped out of the delta into the California aqueduct near Oakland--if most of you are more familiar with the Oakland area--Tracy, for those that are more familiar with California.

Davis:

I would like to ask Mr. Lucas to elaborate on the $6 value for seepage water, whereas the other cost of water--that is evaporation value--is $20. If I read into this correctly, this means that the people that pump seepage water would have to pay $14 an acre-foot for pumping water. If they don't get charged for pumping water, this means that the cost of seepage water is not $6, but $20 plus the cost of removing water that creates a drainage problem. Would you elaborate on this, please?

Lucas:

The Bureau of Reclamation gave us this $6 figure, and it is based upon pumping costs for pumping water back into the canal from immediately alongside the canal.

Vandertulip:

I have an observation for the economic feasibility of lining: I think the paper Mr. Lucas presented certainly indicates justification because there is a need for that water that could be salvaged. Now if we can still salvage water and there is not a need for it, then obviously it is not justified.

SESSION 2.--CHAIRMAN, SOL D. RESNICK

PERCOLATION OF WATER IN SOIL[1]

D. R. Nielsen[2]

The intention of this presentation is to introduce waterflow through unsaturated soils and its relation to the general problem of seepage. Most seepage problems involve both saturated and unsaturated flow, although in the past, greater attention has been given to saturated cases. With the conservation of water being a primary concern, more and more emphasis is being given to the unsaturated cases. For example, unsaturated flow comes quickly into focus when dealing with ground water recharge operations. We find that the soil at greater depths can remain unsaturated and is able to conduct water as rapidly as it passes down through the saturated soil surface. In this case, we know the depth to the water table does not influence the rate of movement. In areas where tile drainage is practiced, the contribution of flow in the unsaturated soil must not be ignored, especially when salinity and reclamation are in question. And in general, for many seepage problems there exists a considerable portion of the soil that remains unsaturated regardless of the length of time seepage occurs.

Let us review briefly what is meant by an unsaturated soil, hydraulic or capillary conductivity, and what are the means of measuring the flow of water through soils. When the soil water pressure is greater than the atmospheric pressure, field soils will generally be nearly water saturated. The water standing in the vertical tube above the soil sample given in figure 1 indicates the saturated condition. When the soil is unsaturated, with a continuous air phase, the soil water pressure is below atmospheric pressure. The water hanging in the vertical tube below the soil sample indicates the unsaturated condition. Thus, the soil water content is at near saturation for positive pressures (above atmospheric) while unsaturated water contents depend upon the negative soil water pressure (below atmospheric). The shape of the curve for negative pressures depends upon the soil. Furthermore, previous levels and rates of wetting and drying affect the position of the curve. Investigators are currently examining the causes of the different shapes and positions of this curve for negative pressures.

Consider another soil sample where two vertical tubes are present instead of one. Let the difference in water level elevation between the two tubes be Δh. For these positive pressures, illustrated in figure 2, there will be a flux of water moving from left to right defined by

$$\text{flux} = K \frac{\Delta h}{L}$$

where L is the sample length and K the hydraulic conductivity. As long as the pressures remain positive, the soil will remain saturated, and the hydraulic conductivity will be nearly constant.

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[1] Contribution from the University of California, Davis, Calif.

[2] Assistant Professor of Irrigation, Irrigation Department.

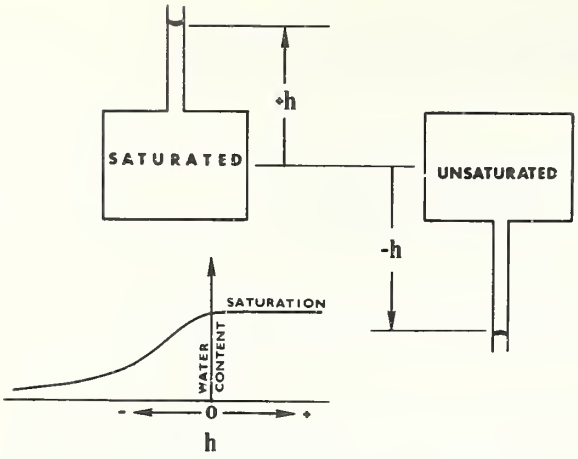


Figure 1.--Schematic drawing relating saturated and unsaturated soil conditions with positive and negative soil water pressures.

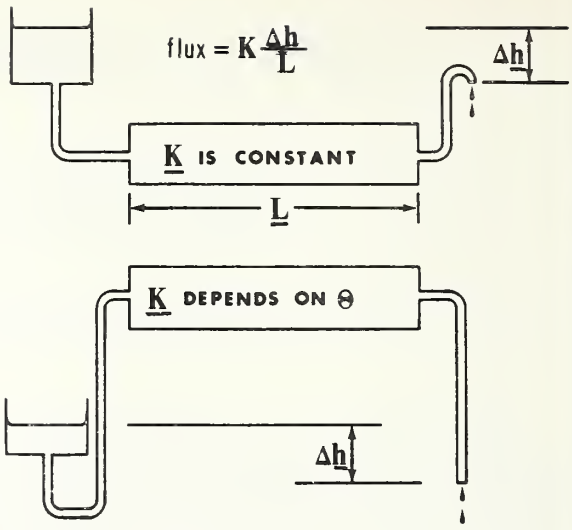


Figure 2.--A sketch of laboratory apparatus used to measure the capillary conductivity K. The upper drawing represents a saturated soil while the lower drawing is for an unsaturated soil of water content θ.

On the other hand, if the vertical tubes impose a negative pressure as given in the lower diagram of figure 2, the hydraulic conductivity K will depend upon the water content. Thus, for negative soil water pressures K is not a constant and decreases as the water content decreases. For unsaturated soils, K is commonly called the capillary conductivity. In the case of vertical soil water movement, not only do the differences in soil water pressure have to be considered but differences in elevation must also account for the gravitational field.

At the West Side Field Laboratory of the California Agricultural Experiment Station located southwest of Fresno, a field experiment was performed to follow changes in both soil water content and soil water pressures within a soil profile. In addition to soil water measurements made while the soil surface was ponded with water for a period of 24 hours, measurements were recorded during a subsequent drainage period of two weeks. Within an irrigated field, a quadrangular plot 100 square feet in area was defined. Within the center of this plot, soil water pressure and soil water contents were measured in a 4-foot-square area and at depths of 6, 12, 18, 24, 30, 36, 48, and 60 inches. Measurements of soil water pressures by tensiometers and soil water contents by a neutron meter were made in quadruplicate at each depth every 30 minutes during the ponded condition. Less frequent time intervals were used during drainage. At the conclusion of the drainage period, a pit was excavated at each edge of the plot and a fifth pit was located in the center of the plot. From these pits soil samples were removed for laboratory determinations of soil bulk density and soil water content-soil water pressure relations.

In figure 3, it can be seen how the soil water pressure changes within the profile after water has been ponded at the surface. Before water was added (0 hours), the soil water pressure had an average value of about -230 mb. throughout the 5-foot depth. The pressure just below the soil surface approaches atmospheric pressure soon after water is ponded. It is interesting to note that during the first 10 hours of ponding, the soil water pressure at the 30-inch depth remains nearly constant while the pressures below that depth continue to change. After 24 hours continuous ponding, the soil water pressures are near 0 (or atmospheric) to only a shallow depth of 12 inches while values of -170 mb. are existing between 36 to 60 inches.

After 4 days of internal drainage with no evaporation at the soil surface, the negative soil water pressures in the top 2 feet of soil have approached 90 mb. indicating a reduction in the soil water content (figure 4). This decrease in the upper part of the profile is accompanied by an increase in soil water at the greater depths illustrated by soil water pressures increasing toward atmospheric pressure. During the subsequent days of drainage, the soil water pressures change in a uniform manner with the least change occurring at the greater depths. The greater depths are losing water while simultaneously transmitting the water draining from the upper portion of the profile.

Although soil water pressure distributions are informative as regards the direction of water movement, it is sometimes more convenient to study water content values directly. In figure 5, for 0 hours corresponding to the time before water was ponded, the initial soil water content distribution is irregular. The water content of the upper portion of the profile is approximately 28 percent, increasing markedly at the 24-inch depth compared with the 18- and 30-inch depths, with the lower portion of the profile at water contents greater than 40 percent. After 5 hours of ponding water at the surface, infiltration has resulted in a water content increase of nearly 20 percent at the 6-inch depth and progressively smaller increases at greater profile depths. These increases may be seen more effectively in figure 6 where only the increase in water content over the initial values is given. For the 10-, 20-, and 24-hour time intervals, the soil water contents are gradually increasing. The saturated values are indicated by the broken line for the 24-hour profile. From these data it is evident that the top 1 foot of soil has the least conductivity or permeability of the profile.

During the first six hours of drainage, the average rate at which water passes the 60-inch depth is about 1.3 inches per day. (See fig. 7.) This water coming principally from the top 18 inches moves through the profile without increasing the water content at the

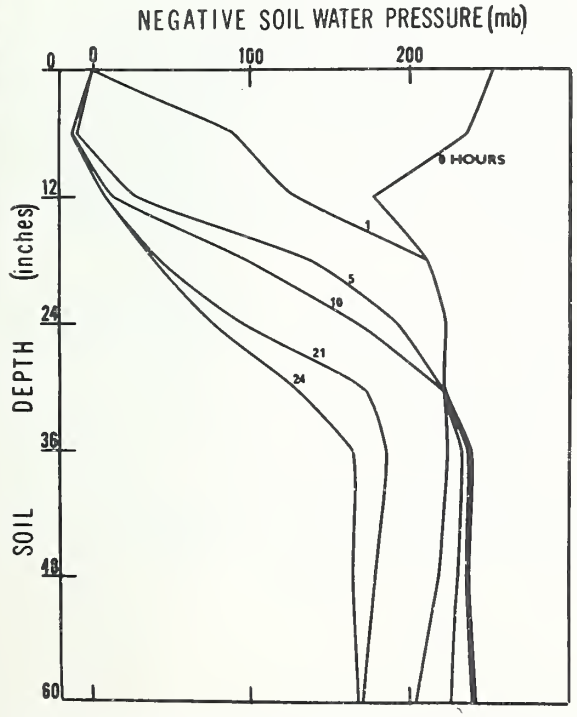


Figure 3.--Negative soil water pressures versus depth after water has been ponded at the soil surface for 24 hours.

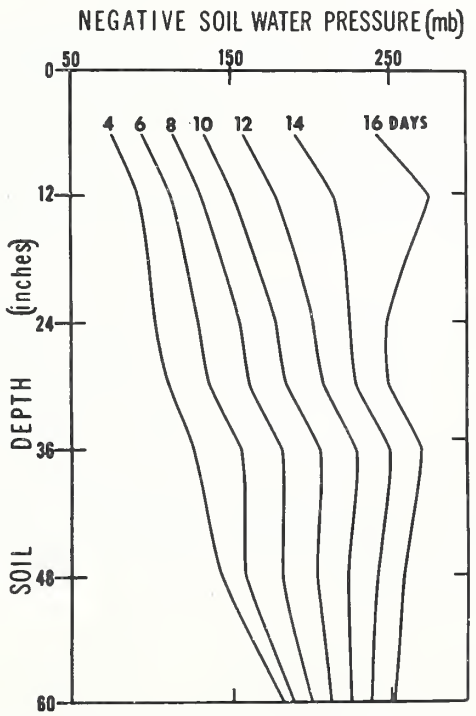


Figure 4.--Negative soil water pressures versus soil depth. The number of days indicates the drainage period since water was ponded at the soil surface.

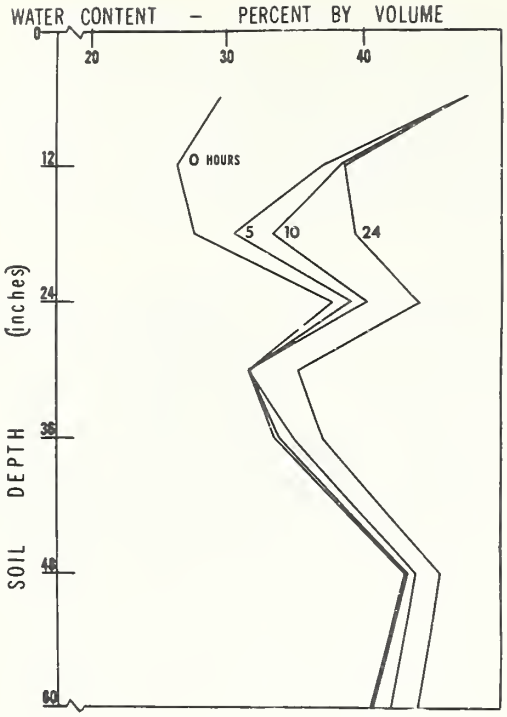


Figure 5.--Soil water content versus depth during 24 hours of continuous ponding at the soil surface. The initial water content distribution before ponding is given by the curve marked 0 hours.

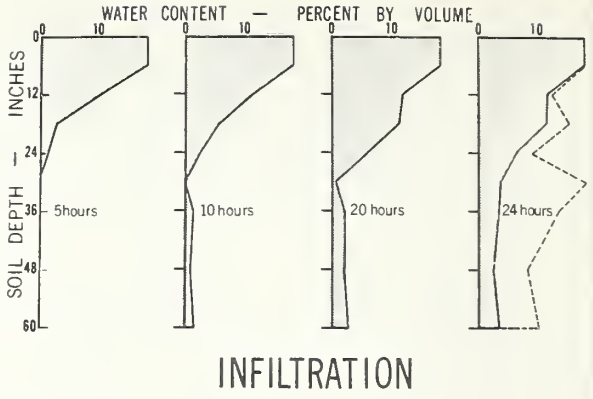


Figure 6.--The shaded areas indicate the increase in soil water content over initial values within the 5-foot profile during infiltration. The broken line for the 24-hour distribution indicates complete saturation.

greater depths. The same may be said for the 6- to 16-hour and the 16- to 46-hour time intervals. During the first 2 days of drainage, the rate of water loss from the 5-foot profile decreases from 1.3 inches to 0.3 inch per day. It is significant to notice in figure 8 that the average rate of water loss from the 5-foot profile is greater than 0.25 inch per day for the next 10 days. This rate is the same order of magnitude as that for water evaporating from a reservoir even though the soil is far from saturation. These results should not be considered peculiar to this soil as similar results have been measured for several other soils. For example, a well-drained Yolo soil near Davis, Calif., was treated in a similar manner and the amount of water passing the 6-foot depth after initial stages of drainage cannot be disregarded. In figure 9, a total of 5 surface inches of water pass the 6-foot depth during 50 consecutive days of drainage.

These two experiments have demonstrated that a substantial quantity of water is capable of moving through unsaturated soils. Whether or not movement is occurring when the soil is unsaturated can be determined by measurements of negative soil water pressures. These measurements cannot be made with open-ended piezometers used for saturated flow. Furthermore, the existence of ground water flow may not be ascertained by measuring soil water contents if the soil is unsaturated and the water contents do not change. Thus seepage losses from canals, surface reservoirs, or underground water reservoirs may be determined by the amount of flow taking place in soils that are far from saturation.

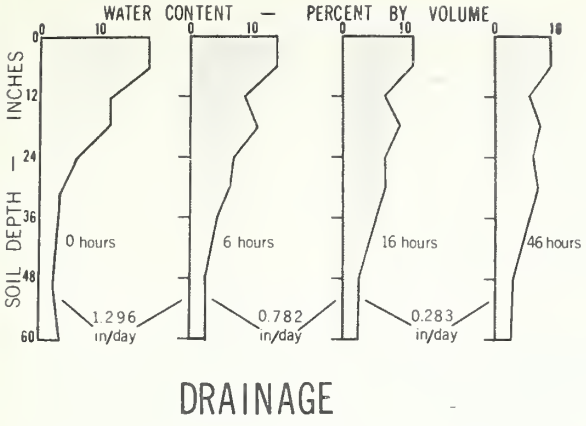


Figure 7.--The shaded areas represent the portions of water added during infiltration that remain in the profile after certain periods of drainage. The values indicated between pairs of profiles are the average rates at which water passes through the 60-inch depth.

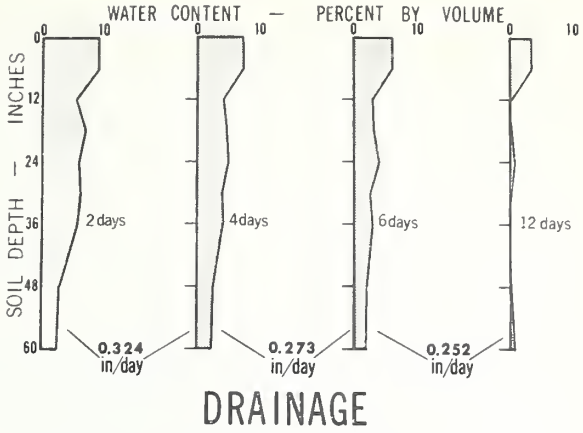


Figure 8.--The shaded areas represent the portions of water added during infiltration that remain in the profile after certain periods of drainage. The values indicated between pairs of profiles are the average rates at which water passes through the 60-inch depth.

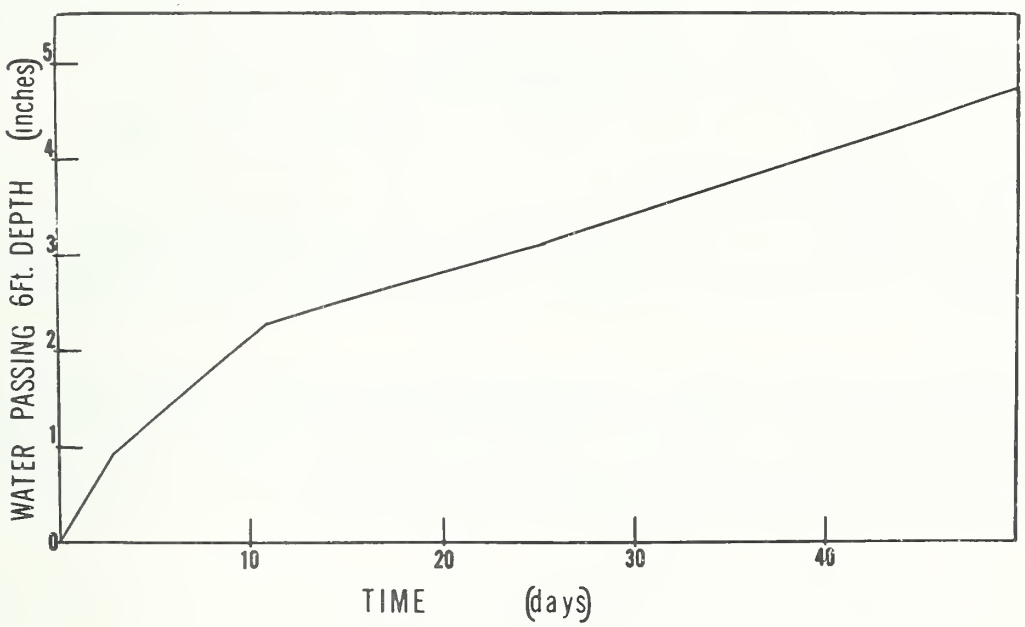


Figure 9.--Cumulative water from internal drainage passing the 6-foot depth of a "well-drained" soil as a result of internal drainage. Zero time corresponds to 3 days after a heavy irrigation.

THEORY OF SEEPAGE FLOW SYSTEMS[1]

Herman Bouwer[2]

BASIC SEEPAGE CONDITIONS

Seepage is a dynamic process that is affected by a number of factors; for instance, the nonuniformity of the soil, cracks, holes, gravel veins, erosion and sedimentation in the canal, biological activity, fluctuation of water tables, alternate wetting and drying of soil for ditches which carry water only intermittently, etc. For an analysis of how seepage is affected in theory by conditions of canal, soil, and water table, it is necessary to introduce a number of simplifications. In this study, the seepage flow systems are reduced to the following three basic seepage conditions (geometry and symbols shown in fig. 1).

A. The canal is in uniform soil. The water table slopes away from the water surface in the canal and becomes horizontal at infinity (or great distance). A highly permeable layer occurs at a certain distance, $\underline{D}_p$, below the canal bottom. A special condition exists when the water table is at or below the top of the highly permeable layer. In this case, the lower boundary of the flow system is a horizontal contour of atmospheric pressure and the flow system is one of seepage to a permeable drainage layer. This condition is referred to as condition A, and the criterion for its existence is $\underline{D}_{w,\infty} \geqq \underline{H}_w + \underline{D}_p$.

B. This condition is the same as condition A, except that an impermeable layer occurs at distance $\underline{D}_i$ below the canal bottom.

C. The canal is lined by a natural or artificial slowly permeable layer which controls the seepage. The underlying material is unsaturated with vertical flow at essentially unit gradient.

Actual seepage may consist of variations and combinations of these three conditions. Although mathematical solutions are available for certain cases of canal seepage, solutions to solve the seepage complex for the entire range of conditions A and B apparently are not. In this study, solutions for conditions A and B were obtained with an electrical resistance network analog. Condition C was treated analytically.

CONDITIONS A AND B

Variables and Results

Variables included in the analog solutions of conditions A and B were the water depth and shape of the canal, the position of the water table as characterized by $\underline{D}_{w,\infty}$, the position of the impermeable layer or highly permeable layer as characterized by $\underline{D}_i$ and

[1] Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2] Research hydraulic engineer, U.S. Water Conservation Laboratory, Tempe, Ariz.

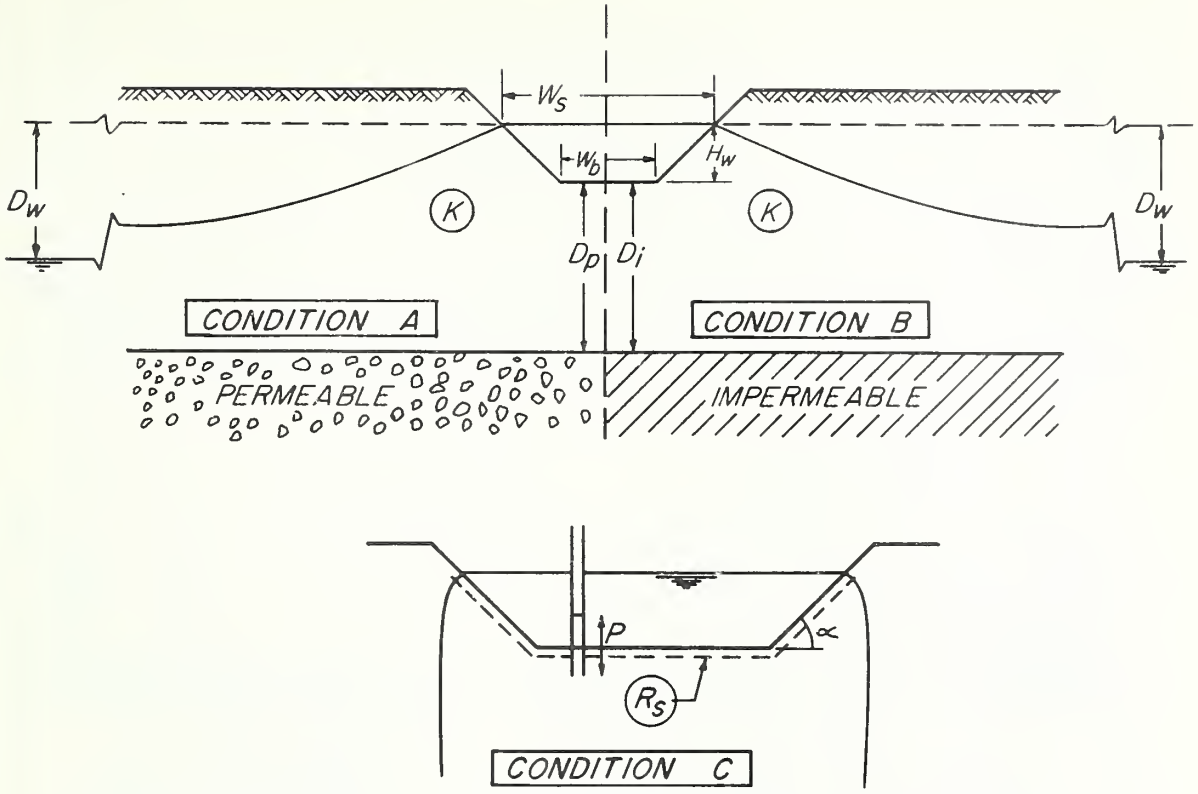


Figure 1.--Geometry and symbols of seepage conditions A, B, and C.

D_p, respectively, the saturated hydraulic conductivity K of the soil, and the unsaturated conductivity characteristics of the soil. The latter were included as certain displacement or critical pressures P_c, assuming saturated conductivity K to the critical pressure and negligibly small conductivity beyond.

The results were expressed in dimensionless terms. Seepage loss was expressed as seepage per unit length of canal divided by the width of the water surface. This factor was then divided by K to obtain the dimensionless parameter I_s/K. Graphs were prepared showing I_s/K as a function of D_w, ∞/W_b for different values of D_i/W_b and D_p/W_b. An example of such a graph is shown in figure 2, which applies to a trapezoidal canal with 1:1 side slopes and a water depth of 0.75 W_b and P_c = 0 (saturated flow only).

For condition A, figure 2 shows that the seepage increases as the depth of the permeable layer decreases. For a given value of D_p, lowering the water table increases the seepage until the water table reaches the top of the permeable layer. At this point, condition A' has been reached and further lowering of the water table has no effect on seepage. Thus, the curves for condition A terminate into the curve describing condition A'.

For condition B, lowering the impermeable layer increases the seepage. Figure 2 shows that if $D_p > 5\ W_b$ or $D_i > 10\ W_b$, the seepage curves are already close to the curve for uniform soil of infinite depth. Thus, permeable or impermeable layers have little effect on seepage if their depth is more than 5 W_b or 10 W_b, respectively, below the canal bottom.

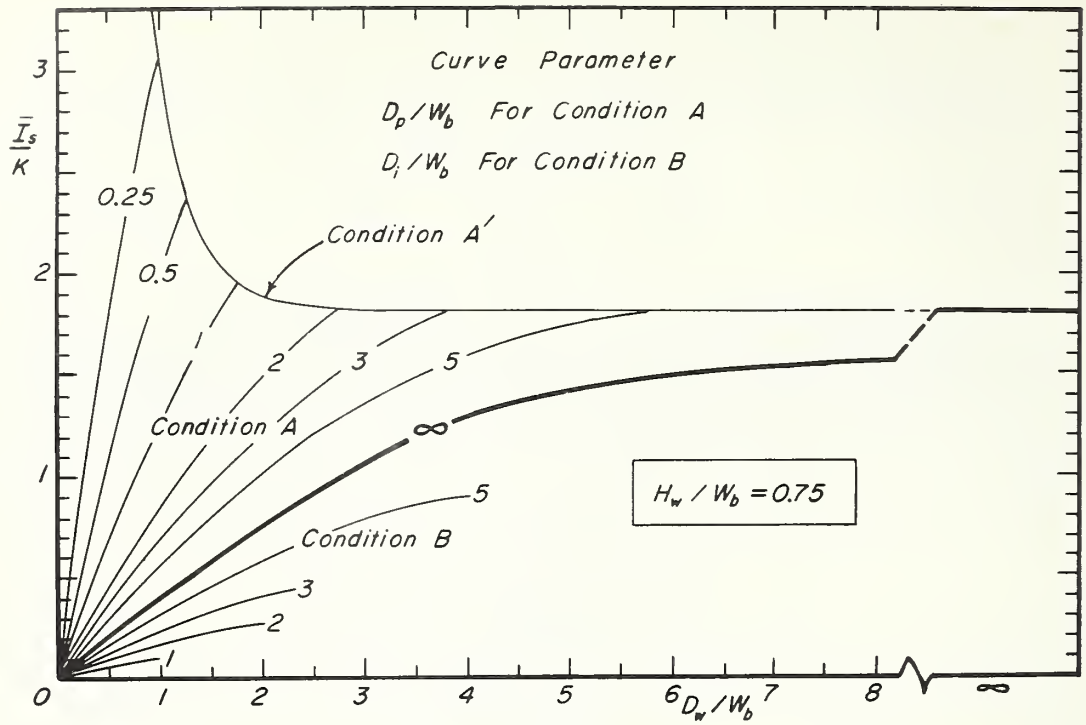


Figure 2.--Results of seepage analyses for trapezoidal canal with $H_w = 0.75\ W_b$.

Lowering the water table increases the seepage until D_w is approximately 5 W_b. The curves in figure 2 show that lowering the water table beyond $D_w = 5\ W_b$ does not affect the seepage appreciably. Thus, if $D_w > 5\ W_b$, the flow system can be considered as one of seepage to an infinitely deep water table or drainage layer. This condition has been treated analytically by Wedernikov (see, for instance, Muskat[3]). For the case of figure 2, the analytical solution yields an I_s/K-value of 1.78, which is nearly the same as the value of 1.81 obtained by analog.

Effect of Unsaturated Flow on Seepage

The effect of unsaturated flow on seepage was studied by introducing arbitrarily selected values of P_c. Figure 3 shows the effect of unsaturated flow for condition A', using a P_c-value of $-W_b/12$. Since the order of magnitude of critical or displacement pressures is approximately -1 ft. of water for a number of soils, this curve is applicable to trapezoidal canals with bottom widths in the range of 12 ft. The effect of the introduction of a P_c-value < 0 for condition A' is an increase in the divergence of streamlines below the canal which causes an increase in the seepage. The flow systems in this case show that positive water pressures exist in a bulb-shaped area under the canal. The rest of the soil-water pressures are negative but the hydraulic conductivity is equal to the saturated conductivity K (in reality, K values slightly less than the saturated K may occur). The broken lines in figure 3 show that inclusion of "unsaturated" flow increases the seepage only slightly. For smaller canals, where P_c may be less than $-W_b/12$, the unsaturated flow can be expected to contribute more significantly to seepage.

[3] Muskat, M. 1946. The flow of homogeneous fluids through porous media. J.W. Edwards, Ann Arbor, Michigan.

For condition B with shallow water tables and shallow impermeable layers, the contribution of unsaturated flow is in the form of a capillary fringe, which provides an extra path for seepage flow to take place. The relative contribution of the flow in this fringe to the flow below the water table appeared to be approximately equal to the ratio of the thickness of the fringe to the average distance between the water table and the impermeable layer. For mildly sloping water tables, the thickness of the capillary fringe is approximately equal to $-P_c$. Thus, if P_c is -1 ft. of water and the distance between the water table and the impermeable layer is 9 ft., the capillary fringe is about 1 ft. high and the flow in the fringe would be approximately 10 percent of the total flow. For shallow water tables (for instance, $D_w, \infty < W_b$) and relatively deep, uniform soils, the effect of flow above the water table on seepage can be ignored. For D_w, ∞-values in excess of 5 W_b, the effect of unsaturated flow can be expected to be similar to that for condition A', as shown in figure 3.

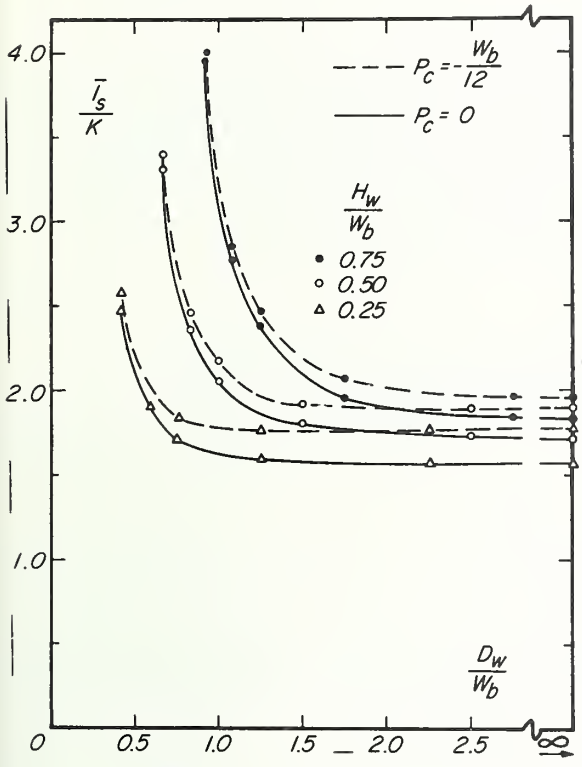


Figure 3.--Results of seepage analyses for trapezoidal canal with 1:1 side slopes under condition A'.

Seepage Distribution

Measurements of the seepage distribution across the canal-wetted perimeter showed that for seepage into deep uniform soils, the point of highest seepage is at the toe of the bank and that the bottom seepage rates are slightly higher than those for the bank. If, for condition A, however, the permeable layer is brought relatively close to the canal bottom, the bottom seepage rates increase sharply and exceed those of the bank considerably. The reverse is true when, for condition B, the impermeable layer is brought relatively close to the canal bottom. In that case, the seepage rates at the bank exceed those at the bottom.

Effectiveness of Bottom Sealing

A rather interesting question in relation to seepage control with waterborne sealants is how much seepage reduction can be obtained if only the bottom of the canal is sealed without treating the canal banks. Analyses of this problem with the analog showed that generally the seepage reduction obtainable in this manner is very small. Substantial seepage reductions by sealing only the bottom can be obtained only if the water depth in the canal is very small compared to the bottom width (for instance, $H_w < 0.1\ W_b$) or if soil nonuniformity or permeable layers close to the bottom cause the original seepage to consist almost entirely of bottom seepage.

CONDITION C

The seepage for condition C can be expressed analytically. Taking the pressure below the clogged layer as equal to the critical pressure of the underlying soil, seepage at any one point of the canal perimeter can be calculated as the total head loss (water depth minus

critical pressure) divided by the hydraulic impedance, $\underline{R}_s$, of the restricting layer. Integrating this for the entire wetted perimeter gives the following expression for the seepage.

$$\underline{\bar{I}}_s = \frac{\underline{H}_w - \underline{P}_c}{\underline{W}_s \underline{R}_s} \left(\frac{\underline{H}_w}{\sin \alpha} + \underline{W}_b \right).$$

This equation applies to relatively thin, slowly permeable linings with sufficient hydraulic impedance to cause unsaturated flow in the underlying soil. The hydraulic impedance of a thin, slowly permeable layer is calculated as the thickness of the layer divided by the hydraulic conductivity of the layer. The dimension of impedance is time, and it can be construed as the time required for a unit depth of seepage to pass through the restricting layer at unit head difference.

REMARKS

The results presented in this paper are part of a more comprehensive study still in progress. Graphs such as figure 2 will be prepared for other water depths in the canal. Critical pressures less than $-\underline{W}_b/12$ will be used to more fully cover the effect of unsaturated flow on seepage. Also, the effect of the shape of the canal will be investigated. A more detailed paper describing the complete studies will be prepared.

RESISTIVITY AND PHOTOGRAPHIC TECHNIQUES FOR SEEPAGE EVALUATION [1]

William B. Shaw [2]

The Department of Water Resources of the State of California is conducting studies of river seepage in the Sacramento Valley. In making these studies, two relatively new techniques have been used to help determine the areal extent of areas that are waterlogged under various conditions of river stage and stage duration due to seepage. This paper relates some of our experiences in utilizing the tools of infrared photography and electrical resistivity measurements for this purpose.

The Sacramento Valley Seepage Investigation began in October 1959, as a result of concern in the Legislature over the possibility that additional seepage might be induced through operation of the State Water Facilities. The key water conservation feature of the State Water Facilities is Oroville Reservoir on the Feather River, located just upstream from Oroville, which will have a capacity of nearly 3 1/2 million acre-feet.

In 1959, the California Legislature established the policy that the cost of alleviating seepage problems which may arise from construction and operation of a water project shall be borne by the project. In addition, the Department was enjoined by the Legislature

[1] Contribution from California State Department of Water Resources, Sacramento.
[2] Senior engineer, Department of Water Resources.

to anticipate seepage problems which may arise from future construction and operation of water projects and to include plans for the solution of seepage as part of the project development.

The objectives of the Sacramento Valley Seepage Investigation are:

1. To determine the location and extent of areas along the leveed watercourses in the Sacramento Valley affected by seepage;
2. To determine the effects of the various factors influencing seepage and to evaluate the possible future occurrence and extent of seepage;
3. To estimate the economic effects caused by present seepage and by possible future seepage;
4. To estimate the costs and effectiveness of corrective works needed to mitigate seepage damage;
5. To establish a basis for measurement of damage due to operation of upstream water development projects and to establish an equitable basis for allocating costs of any corrective works needed to mitigate this damage; and
6. To indicate if there is a need for detailed studies leading to authorization of definite projects to construct works for seepage mitigation, and to determine which projects merit consideration.

A final report is scheduled for publication at the conclusion of the investigation in June 1965.

The Sacramento Valley, together with the San Joaquin Valley, forms the great central valley of California. The valley lies between the Sierra Nevada Mountains to the east and the Coast Range to the west. The Sacramento and San Joaquin Valleys merge in a delta region traversed by many interconnecting sloughs.

The Sacramento Valley is relatively narrow in its upper end above Chico. In this upper portion, the river has sufficient capacity to carry major flows below the level of the adjacent ground, and river seepage is not a problem. South of Chico the valley widens and the slope of the river is flatter. The outflow from the river during flood stages has deposited sediments, building up strips of land sloping away from the river banks. These "natural levees" range from a few feet to 20 feet or more above the adjacent land and slope toward two main shallow troughs, which in turn slope upward toward the base of the foothills. The two main troughs are broken into smaller basins by built-up banks of tributary streams, particularly the Feather and American Rivers. The Sutter Buttes, a remnant of a volcanic plug, stands in the center of the valley.

Flood protection and reclamation of overflow lands began about 1851. Now, with a highly developed flood control system, flood waters resulting from snowmelt in the mountains and intense rainfall on the foothills are contained by levees. When floods occur, the water levels in the rivers and in the flood control bypasses are above the surface of the adjoining lands. Water percolates from these watercourses, under and through the levees, saturates the soil on the landside of the levees and, in some areas, ponds on the land surface. When high flows persist, considerable damage is caused by seepage.

The first objective of our investigation is to determine the location and extent of areas along the streams and bypasses affected by seepage under differing flow conditions. It is in meeting this objective, so basic to subsequent objectives, that we have been utilizing infrared photography and resistivity measurements.

Infrared film is sensitive to a wider range of wave lengths than is panchromatic film. In actual practice, the short end of the spectrum must be "blanked out" by the use of a suitable filter. As is astronomical photography, by knowing the values of the particular film and the coefficient of reflectance of various surfaces, one may determine mathematically the type of material photographed. Selected land photography is used in a variety of military and scientific work.

For our work in identifying and mapping areas waterlogged by seepage, we first went to aerial photography because of the extent of the area we needed to cover (about 500 miles including both sides of the watercourses) and the time required to train competent seepage observers. It generally takes several years to become proficient in this art. The first year many of our observers are eager and ambitious and they see "seepage" everywhere. The second year they have been "stung" and they see very little "seepage." By the third year, our observers settle down. By then they are sufficiently skilled to watch for telltale clues such as drying cracks, shine or dullness of wet spots, the changing extent of wet areas, and sponginess. The liberal use of a shovel is often beneficial.

Why all this difficulty? Because our seepage occurs during prolonged periods of high river stage, and these occur only during our rainy season. Unless one is very fortunate when trying to observe seepage, there tends to be ponded drainage water adjacent to and intermixed with seepage. Notable cases of drainage causing some confusion are observed in the orchards and sugar beet lands that seem to stay wet for long periods following a rain. We speculate that orchards stay wet due to a "plowpan" and that the heavy equipment used in harvesting causes a similar phenomenon where sugar beets are grown.

In addition to difficulties are interpretation, it is difficult to plot the extent of waterlogging, as access is limited to the levees and a few crossroads. It is slow going on foot when one has to walk into the seepage areas. The observer would probably sink quite deeply into the waterlogged soil.

Aerial photography seemed an answer to many of the difficulties encountered in observing seepage. The initial photographs were taken with panchromatic film, using a minus-blue filter to eliminate the scattered blue light reflected from the ground. Use of these photographs gave superior delineation of seepage areas, since the horizontal extent of waterlogging could be mapped from the photograph. However, some waterlogged areas were difficult to define because of the lack of contrast between areas where the soil was saturated or nearly saturated and areas where the soil was waterlogged.

Many of the areas where this lack of contrast was observed were fallow lands, where nearly saturated soil had much the same light-reflecting characteristics as adjacent waterlogged areas.

In order to sharpen the contrast between waterlogged and nearly saturated soils, we are now using infrared film with a light-red filter. The resulting spectral band gives a sharp contrast between waterlogged areas and adjacent saturated areas. Because water has a high degree of absorption of infrared frequencies, the waterlogged areas appear quite dark in photographs.

The following photographs are of small areas we have selected for collection of detailed information for model studies.

Heavy rainfall from February 6 through 19, 1962, saturated surface soils in the Sacramento Valley and caused drainage water to collect in depressed areas.

Figure 1, a photograph taken with black-and-white film, gave only a low order of contrast between areas waterlogged by seepage and adjacent partially saturated areas, although seepage conditions were maximum at the time the photograph was taken.

Figure 2, a photograph taken on February 26, 1962, demonstrates the high-order contrast obtained with infrared photography using a minus-blue filter. Care is needed, however, in ordering this type of photography because the photographer may have new equipment called a Log Etronic printer that makes a sharper and more pleasing picture by balancing the tone of the photograph. This reduces the contrast. These particular photographs were developed so as to obtain contrast. If the new developing equipment is used, a red filter may be needed to get this degree of contrast.

Figure 3, a photograph taken on May 30, 1962, was made with infrared film and a minus-blue filter. At this time there had been no heavy rainfall for 2 months and the river was below critical seepage stage. The darker areas on the photograph, for the most part, were under irrigation, but some of the areas are dark because of the reflectivity of the soil or vegetative cover. This particular series of photographs was obtained for control in interpreting other photographs taken during the seepage season.

Figure 4, a photograph taken on October 19, 1962, was made after an unseasonal, high-intensity rainstorm. Soils were uniformly saturated, with some ponded water in the depressed areas. The Sacramento River reached critical seepage levels, but rapidly declined, and no waterlogging is shown in the photographs. This series of photographs should provide excellent control for the photographs we will obtain during times of seepage.

We presently have photography ordered, since recent river flows have created extensive seepage. Our problems in taking these photographs illustrate the principal drawback of aerial photography in observing seepage conditions. A persistent series of low ground fog and clouds has made it impossible to get aerial photographs exactly when we wanted them--and we have been waiting over a week for a break in the weather--since it is desirable to get them at about the maximum extent of waterlogging and to get out in the field to correlate and interpret the photographs before the seepage areas dry out too much. It is obvious that it is difficult to obtain the photographs at the optimum time.

ELECTRICAL RESISTIVITY

Aerial photography is, of course, limited to surface measurement and interpretation of surface data. Measurement of the electrical resistivity of the top 20 feet of the soil can be utilized to advantage in conjunction with aerial photography. Each supplements the other; together they form a valuable aid for locating seepage.

The purpose of electrical resistivity is to outline seepage areas on the basis of the contrasting electrical resistivities of clayey and sandy soil. Sandy soil has a higher electrical resistivity than clayey soil. The electrical-resistivity method was used mainly

Figure 1.--Black and white film showing the low-order contrast between areas waterlogged by seepage and adjacent partially saturated areas.

Figure 2.--Infrared film showing the high-order contrast between waterlogged areas and adjacent partially saturated areas.

Figure 3.--Infrared film showing high-order contrast between areas under irrigation and adjacent dry areas.

Figure 4.--Infrared film showing appearance of area with all areas saturated but with very little standing water. Note the lighter appearance of the river, as compared to the other two infrared films, due to tone balancing.

as an economy measure, because the alternate method of soil sampling--by drilling at half-mile intervals or less along 500 miles of river and bypass levees--would be time-consuming and expensive.

The conventional Wenner configuration of electrodes is used to obtain the electrical resistivity by measuring the potential difference within the soil between two potential electrodes located between two current electrodes. A dry-cell battery supplies the current and a Wheatstone bridge measures the potential difference. In the Wenner method, the spacing between the electrodes is proportional to the volume of the soil being measured. The electrical-resistivity value is a function of the spacing of the electrodes and the electrical resistance of the soil, which is determined by the potential difference and the current.

The electrical resistance of the soil is dependent on the water content and the chemical characteristics of the soil. Along the Sacramento River the electrical-resistivity values range from 20 ohm-feet for clayey soils to over 100 ohm-feet for sandy soils. The electrical-resistivity values are obtained from a horizontal traverse at each location at a constant depth of 20 feet. When the electrical-resistivity values are plotted to form lines of equal resistivity, the area where resistivity is greater than 100 ohm-feet is noted as a potential seepage area. During a seepage period, when the soils become saturated, the resistivity values decrease due to the higher water content; but because the resistivity of river water averages considerably more than 100 ohm-feet, sufficient contrasts in electrical-resistivity values remain between clayey and sandy soils in spite of increased water content. A constant depth traverse of 20 feet was made because of the general observation that 20 feet of clayey soil overlying a pervious substratum would generally provide a barrier to underseepage flow. Electrical-resistivity methods enabled the differentiation to be made between river seepage, drainage from rainfall, and drainage from seepage. This differentiation aided in the interpretation of aerial photos and field observations.

The outlining of seepage areas by electrical resistivity is very rapid. By having the conducting wires of the electrical-resistivity equipment systematically set at constant lengths, one man can move in, measure, and move out in a few minutes. In the Sacramento Valley Seepage Investigation, the high speed and low cost of making electrical-resistivity measurements have resulted in quickly and accurately outlining seepage areas with a minimum of drilling.

The combined use of aerial photography with electrical-resistivity measurements has enabled the Sacramento Valley Seepage Investigation to effectively and economically obtain the field data necessary for seepage computations. A final report on this work will be included in the report scheduled for publication at the conclusion of the investigation in June 1965.

DISCUSSION

Tuesday afternoon - First Session

Freeland:

I have a question for Mr. Bouwer. He said that as the depth increased in a canal, the seepage rate increased, and I was wondering if he was speaking of total seepage, reflecting the increased wetted perimenter or merely the increase in the seepage rate.

Bouwer:

The rate referred to is the total seepage loss per unit length of canal.

Freeland:

Including the effect of the added wetted perimeter?

Bouwer:

That's right. For the trapezoidal canal with 1:1 side slopes referred to in this study there is an almost straight line relationship between water depth and total seepage per unit length of canal (for conditions A, A', and B). Obviously, however, seepage is not zero when the water depth is zero, so that the line in a graph of seepage vs. water depth does not go through the origin. For condition C, seepage from a trapezoidal canal varies approximately with the square of the water depth.

Haise:

I would like to ask Don Nielsen if the area which was flooded in his studies was covered by some sort of plastic or membrane?

Nielsen:

Yes, it was covered. There was essentially no evaporation at the soil surface on both of those experiments.

Busch:

I have a question for Don Nielsen. One preirrigates cotton and such crops to store water at depth in the soil profile. Now, are you storing enough this way, or are you continually losing it as in the case of the deep water table?

Nielsen:

This experiment was performed on a soil that had previously been wet several times, so we were dealing with a wet profile already. You may have noticed that after 2 weeks of drainage the water content values had returned almost to those at the beginning of the experiment. This is the expected result because the drainage period, after a thorough soaking immediately before the experiment, was 2 weeks, also. As regards to storage capacity, water-content changes within the profile could be measured indefinitely. We actually continued the measurements for more than 4 weeks and during that time the water content continually decreased. There is no constant amount that can be stored in the profile. An additional benefit that is derived from preirrigation, especially in the area discussed, is that of leaching salines from the soil profile. In the wintertime when evaporation is low, if during each day a quarter of an inch of water is passing through the profile without filling it, a significant amount of salt is being leached without increasing the load on the drainage system excessively.

Shockely:

I think that the key point that you brought out is that after the extreme drainage, you only had negative pressure of about one-fourth of an atmosphere. Is this right?

Nielsen:

That's right.

Shockely:

This preirrigation applied on a dry soil would store a lot of water before you ever got up to a negative pressure as low as this.

Nielsen:

That's right. On the other hand, there are several generalities which Herman Bouwer can draw from his analysis. However, in any case, it is necessary to define the magnitude of the flow rate existing at the greatest depth of the soil profile that is considered insignificant. In addition, the total time of drainage must be considered relative to the problem.

V. Myers:

I would like to direct a question to Mr. Shaw regarding photography: Were you able to detect differences in reflectance patterns in various types of vegetation that would help you in making your interpretation of seeded crops?

Shaw:

We actually haven't done that yet; however, I think that you might. Some of the more noticeable ways to identify seepage areas is that long after the area has seeped, there is a weed pattern, and in some of these areas, you get a sharp contrast if the weed content is great enough.

Rice:

I would like to ask Mr. Shaw if refraction seismograph methods were considered, in addition to your resistivity measurements.

Shaw:

We did consider it; however, resistivity measurements seem better suited to our needs.

Nielsen:

Mr. Shaw, the unit "ohm-feet" is strange to me. Not having made those measurements, I would like to know the relation between soil-moisture tension, or negative pressure, or permeability, and that resistance unit. Another question is: Instead of using resistance measurements, which might be influenced by salt content and other factors, would you use small pressure transducers buried at the 20-foot depth to measure pressures directly?

Shaw:

We have thought of pressure transducers but we haven't installed any. Resistivity measurements do not correspond closely to soil-moisture tension or to permeability. There is a large range of variation between the soils in any classification and in any given locality. However, the soils along the Sacramento River appear to be fairly close to saturation throughout most of the year. We use resistivity to determine boundary locations and a general idea of structure. For permeability we use soil samples, and for moisture content we use a neutron probe.

Nielsen:

On the basis of the answer, how did you pick your 100 ohm-feet?

Shaw:

Experience. Actually, it's not going to stay at a 100 ohm-feet for the entire river; some areas will go down to 75. We try to map out to 25 ohm-feet; and when you are at 25, you are pretty definitely in clay material. Resistivity could be a lot higher than 100 ohm-feet, but at about 100 ohm-feet you can't have a clay layer of any great extent. A clay layer of 20 feet in depth generally may be considered to be very nearly an impermeable barrier for the purposes of our studies.

Davis:

I have a question for Don Nielsen. Quite a few of our soils for reclamation involve soils with extreme discontinuities, cracks particularly, such as mud flows and other organic soils. Would you care to extrapolate what you mentioned today? On those soils that have these extreme discontinuities, and where it is very difficult to get any salinity reclamation on these at all, you have unsaturated flow?

Nielsen:

Are you speaking about soils that are extremely dry or wet?

Davis:

Either case.

Nielsen:

Whenever cracks exist to the soil surface, water goes right down them if free water is at the surface. In a few areas--some soils along the Missouri bottom are an example--a major portion of the water enters through surface cracks during irrigation. With these clayey soils I would expect negative soil water pressures most of the time.

Davis:

Do you get much water movement through the soil mass itself for reclamation? If not, how can you correct this?

Nielsen:

It is possible, but not very likely under these conditions. One must consider not only the volume of water passing, but the quantity of salt that is moving. In soil that is highly permeable, large quantities of water might pass through it, but this is no guarantee that salt is being removed in an efficient manner. On the other hand, if the water moves more slowly as is the case in a less permeable soil, the leaching process is more efficient. This efficiency is caused by the greater mixing of the salines in all pores when the flow is reduced. In all cases, water must move through and out of the profile, but it does not necessarily follow that salt movement is proportional to water movement.

SOME INFLUENCES OF SOILS AND WATER TABLE DEPTHS ON SEEPAGE FROM LINED AND UNLINED CANALS[1]

A. E. Reisenauer and R. W. Nelson[1]

INTRODUCTION

Hydrology research conducted by the Hanford Laboratories of the General Electric Co. includes studies of saturated and partially saturated flow in porous media. These studies are directed primarily toward obtaining a better understanding of radioactive liquid waste movement rates and distribution patterns in the vadose and saturated zones. Although this research is related principally to the atomic energy field, the analytical techniques and solution methods that have been developed have application in solving other practical hydrologic problems.

Seepage losses from unlined and earth-lined canals, a subject of considerable interest to the arid West, is an area of investigation which is amenable to rational analysis. The development of methods for the theoretical evaluation of canal seepage losses, as affected by the several independent variables, is particularly desirable since field experiments are often costly to perform and difficult to control. Measurements of the efficiency of canal linings in controlling seepage loss are needed to make economic analyses of such linings. Usually the before and after seepage flow rates and flow patterns are required to determine the supplemental drainage of adjacent farm lands. The subject of farm land drainage in itself poses many problems to the canal designer.

Predicting flow rates and patterns can be accomplished through solution of the boundary value problem describing the flow system of interest. Flexibility in solution methods is required to solve the diversity of practical problems. To achieve this flexibility, a variety of mathematical methods were judiciously combined to form the basis for the Hanford-developed computer program, "Steady Darcian Flow in Soils" which will be discussed briefly. Several typical steady-state flow problems will be presented to illustrate the effects of depth to water table, earth canal linings, and other variables on seepage losses.

BASIC EQUATIONS

The basic equations and implied assumptions are derived and discussed in detail elsewhere by Schiedegger (2, pp. 91-111)[2] and others. It will be sufficient here to state that the generalized Darcy expression was used with the proper equations of state and conservation of mass to give the following equation in Cartesian coordinates:

$$\underline{K}\left[\frac{\partial^2 \underline{\phi}}{\partial \underline{x}^2}+\frac{\partial^2 \underline{\phi}}{\partial \underline{y}^2}+\frac{\partial^2 \underline{\phi}}{\partial \underline{z}^2}\right]+\frac{\partial \underline{K}}{\partial \underline{x}}\frac{\partial \underline{\phi}}{\partial \underline{x}}+\frac{\partial \underline{K}}{\partial \underline{y}}\frac{\partial \underline{\phi}}{\partial \underline{y}}+\frac{\partial \underline{K}}{\partial \underline{z}}\frac{\partial \underline{\phi}}{\partial \underline{z}}=0 \qquad (1)$$

[1] Contribution from General Electric Co., Geochemical and Geophysical Research Chemical Laboratory, Hanford Laboratories, Richland, Wash. Work performed under Contract No. AT(45-1)-1350 for the U.S. Atomic Energy Commission. HW-SA-2890.

[2] Underscored figures in parentheses refer to References at end of paper.

In addition to equation 1, three other relationships are used for partially saturated flow systems. They are:

$$\theta = \underline{F}(\underline{P_c}) \tag{2}$$

$$\underline{K} = \underline{f}(\underline{P_c}) \tag{3}$$

$$\underline{P_c} = \frac{-\underline{P}}{\rho\,\underline{g}} = \underline{z} - \emptyset \tag{4}$$

where

$\underline{F}(\underline{P_c})$ is the functional relationship of moisture content to capillary pressure head.

$\underline{f}(\underline{P_c})$ is the functional relationship of moisture conductance to capillary pressure head.

$\underline{K}$ is the scaler capillary conductivity of the soil. ($\underline{K}$ is a function of capillary pressure head, $\underline{P_c}$, during partial saturation and becomes the hydraulic conductivity or permeability at saturation.)

$\emptyset = \frac{\underline{P}}{\rho\,\underline{g}} + \underline{z}$ is the piezometric head, hydraulic potential, or potential function.

$\underline{P}$ is the hydraulic pressure.

$\underline{P_c}$ is the capillary pressure head.

$\underline{P_e}$ is the capillary pressure of the soil where air begins to enter the pore thereby reducing the capillary conductivity.

$\underline{z}$ is the position head or potential energy due to location in the gravitational field.

θ is the moisture content on a volume basis.

ρ is the fluid density.

$\underline{g}$ is the gravitational scaler.

$\underline{x}, \underline{y}, \underline{z}$ are the coordinate directions with $\underline{z}$ oriented parallel to the earth's gravitational field.

SOLUTION METHOD

Finite difference techniques were used to reduce the partial differential equation to a system of simultaneous equations. Central differences were used throughout as estimates of the first derivatives while the usual combined forward and backward differences were used to represent the second partial derivatives. A modified Gauss-Sidel iterative method was used because of the nonlinearity of the equation. Detailed discussion of the solution method and the appropriate equations are presented elsewhere (3).

The Computer Program

The computer program, written for an IBM 7090, can handle one-, two-, and three-dimensional and axisymmetrical problems with up to 8,000 grid points. Four 8,000-word matrices are stored in immediate access memory. This results in a very efficient computation with an average of about 400 grid-point calculations per second for partially saturated flow problems. Even at this rate very large problems may require 30 minutes or more of computer time, while smaller problems will require 5 to 10 minutes.

A full range of boundary conditions is available for use in a system sufficiently flexible that every grid point can be individually controlled for boundary type. Soils may be heterogeneous or homogeneous since a soil identification matrix is stored in the computer memory. Also, soils may be repeated in the problem as many times as necessary to describe the underlying soil pattern.

Descriptive data for up to 15 different soils can be included in any one problem. These data, which are the functional relationships $f(P_c)$ expressed in equation 3, must be obtained experimentally for the soils in question. The capillary conductivity-capillary pressure relationships for three typical Hanford soils are shown in figure 1. The information plotted on logarithmic coordinates is described mathematically by dividing the curve into parts to conserve computer storage. The parts consist of a constant value up to the entry pressure, $\underline{P_e}$, a table of values to round the curve, and an exponential fit to describe the straight line portion.

The same program can be used to calculate the moisture content from the final potentials by using the functional relationship, $\underline{F(P_c)}$, of equation 2. Figure 2 shows a moisture content-capillary pressure relationship for the same soils as used in figure 1.

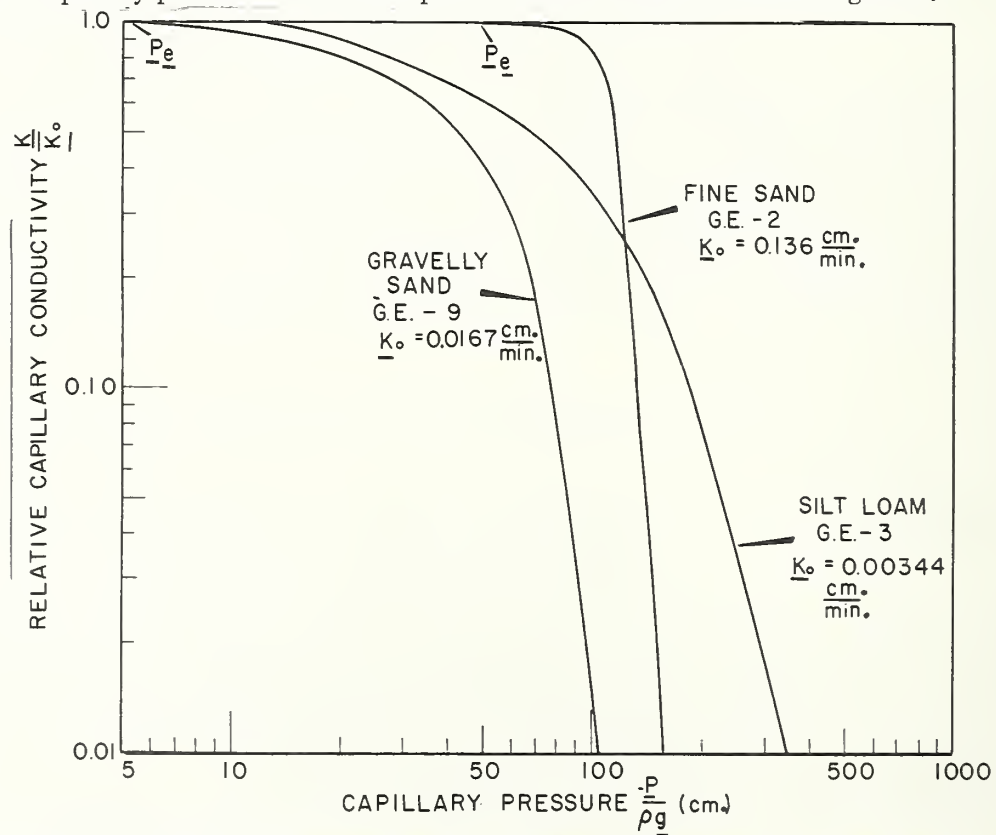


Figure 1.--Capillary conductivity-capillary pressure relationships for typical Hanford soils.

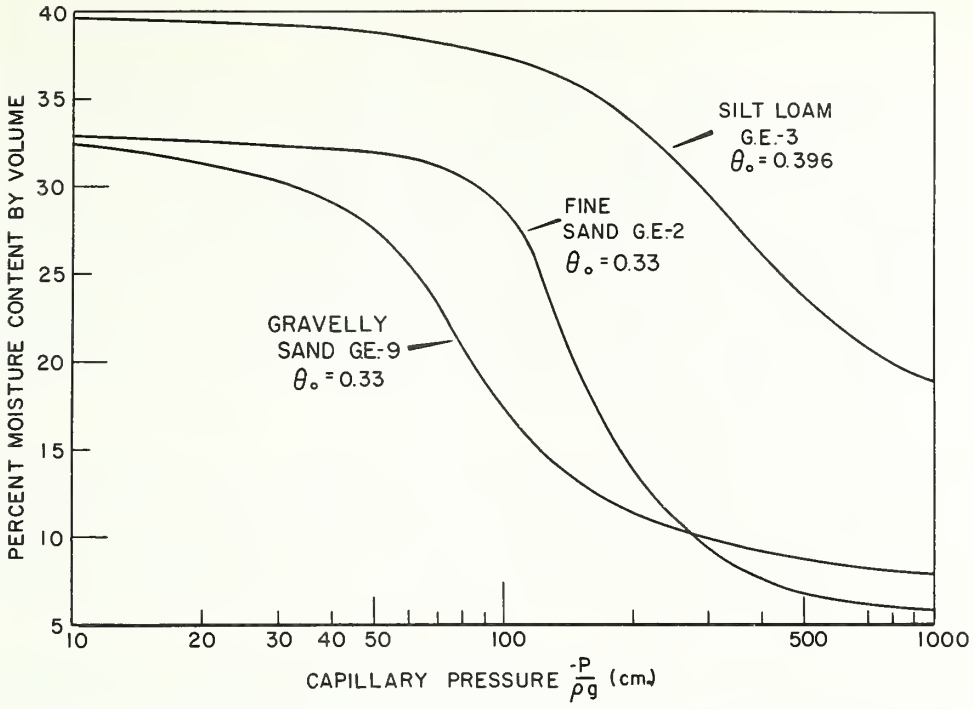


Figure 2.--Moisture content-capillary pressure relationship for typical Hanford soils.

Convergence and Stability

Inherent in the nonlinearity of the basic equation are the problems of convergence and stability of the iterative solution method for a given set of boundary conditions. Several techniques are used to overcome these problems. In the process of iteration the capillary conductivity values are calculated from the potentials; thus, when a new potential is calculated the capillary conductivity is lagging by one iteration. For a better estimate of $\underline{K}$, the values of the minimal surrounding nodes are averaged. In our experience the iterative solution of partially saturated flow problems will not converge without the averaging of $\underline{K}$ values.

To accelerate convergence and save computer time, an optimum overrelaxation factor is used. This internally computed, optimum factor described by Young (5) works extremely well in saturated flow problems where the basic equations reduce to the linear form. In partially saturated flow an upper limit of 1.2 is imposed to maintain stability; however, this is not always sufficient. A method is incorporated in the overrelaxation calculation for detecting instability. When instability occurs, the program uses an equation derived from experience that returns to the program an overrelaxation factor less than one. The result is a tendency toward averaging of the potential values which effectively dampens the oscillations.

Another method of reducing instability and speeding convergence is related to an optimum order in which the system of equations is solved. A real understanding of such convergent sequences must await the results of Steward (3), and others working in this area of applied mathematics. However we have found that instabilities can be controlled by using a movable initial point which in effect reorders the iteration through the system of simultaneous equations. When the initial point is moved to the point of greatest disturbance, instabilities are dampened and the problem ordinarily converges. The judicious use of these various controls has enabled us to overcome instabilities encountered in problems worked on to date.

Several typical canal seepage problems were solved using these computer techniques. From such solutions the effects of water-table depth, canal size and shape, soil types, and various soil linings on seepage losses can be evaluated. The results presented are restricted to a study of a single canal size and a limited range of soil lining permeabilities, since in waste disposal applications the flow pattern and velocities are of more interest than the seepage losses. However, tabulated results for a variety of canal sizes and lining materials could be readily obtained. Figure 3 shows the streamlines and equipotential contours for a flow system in which the source is an infinitely long canal, 15 feet wide and 5 feet deep, with a depth to ground water of 39 feet. The moisture content pattern in figure 4 shows the degree of partial saturation of the flow system. The volume fraction moisture content line of 0.33 represents saturated soil.

The importance of considering partially saturated flow is emphasized when rates of seepage loss calculated under that condition are compared to results obtained when saturated flow is assumed. The seepage loss for the above case of partially saturated flow is 26.1 $ft.^3$/day/linear ft. of canal; whereas a loss of 21.8 $ft.^3$/day/linear ft. is calculated from an analysis reported in Muskat (1, pp. 331-336) for the same soil and canal.

Additional flow systems with other water table depths were solved using the same soil and canal geometry. The data in figure 5 show the relationship of seepage loss of water table depth. When the depth to the water table is great enough and negative pressures occur beneath the canal, the dependence of flow on water table depth is greatly decreased. The analysis from Muskat (2) indicates that the loss is independent of the depth to water.

Figure 3.

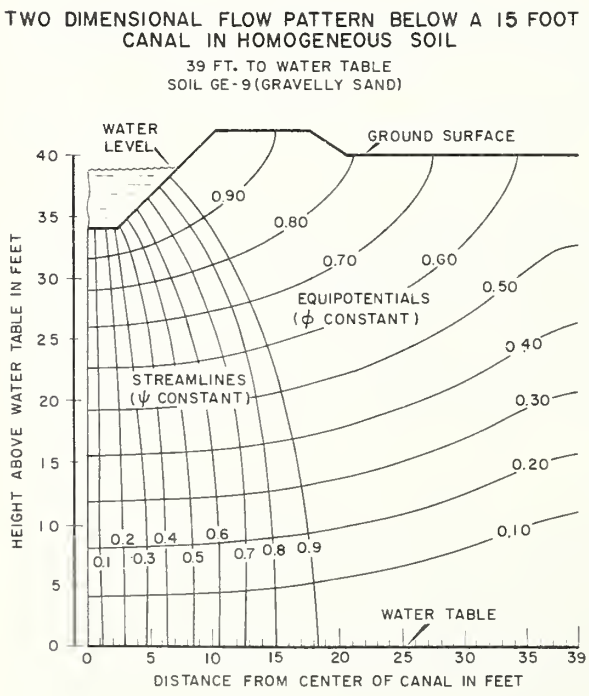


Figure 4.

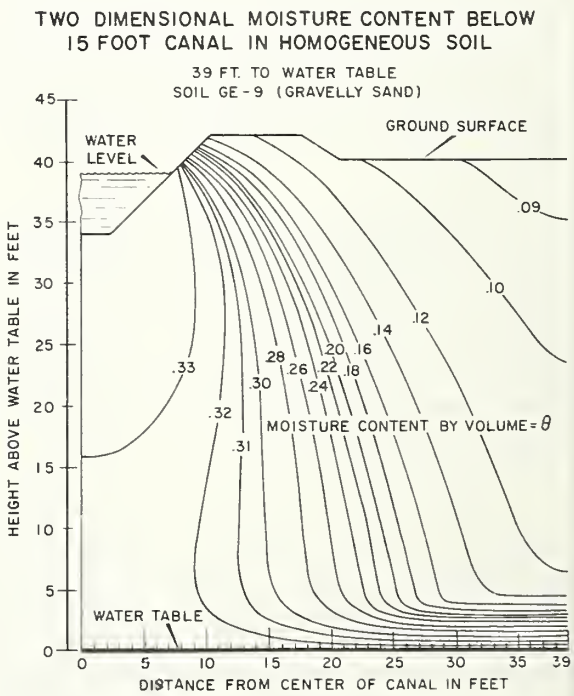
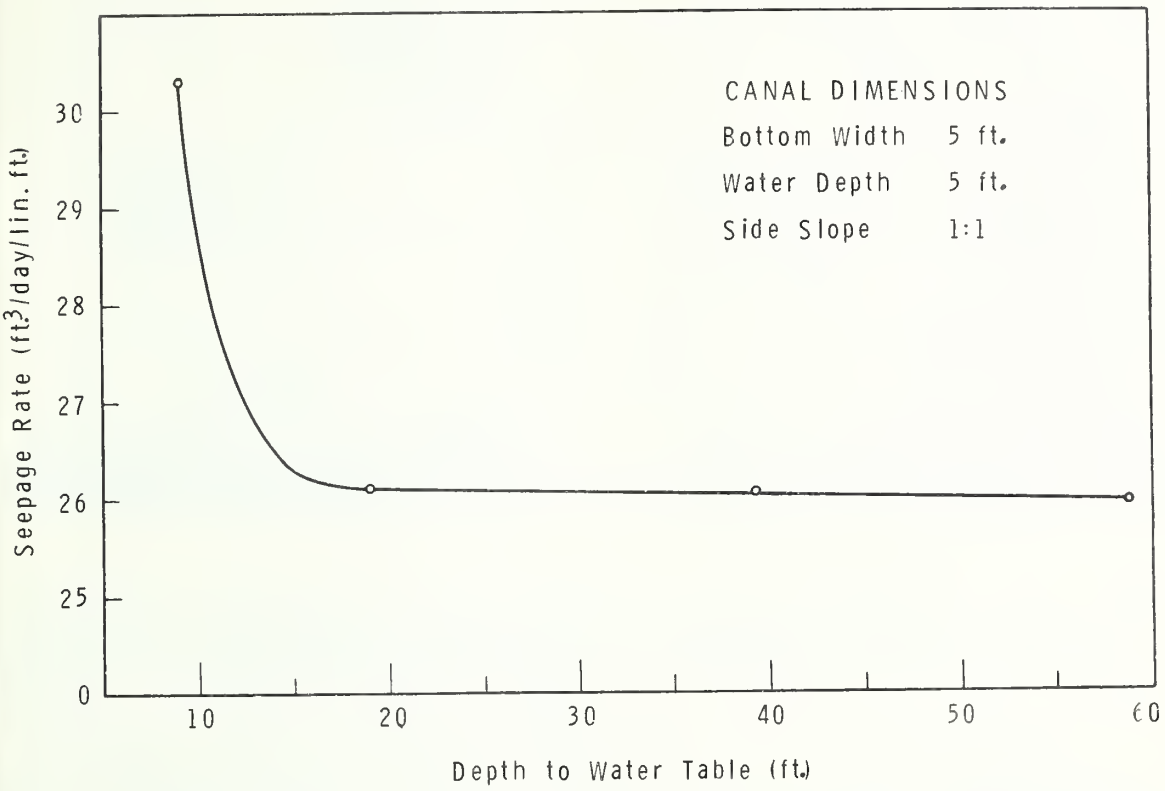


Figure 5.

RELATIONSHIP OF SEEPAGE RATE TO WATER TABLE DEPTH FOR PARTIALLY-SATURATED FLOW IN SOIL GE-9

($K = 0.789$ ft.^{3}/ft.^{2} Day $\frac{\text{ft.}}{\text{ft.}}$)



Effects of Earth Lining on Seepage

The effects of earth lining on seepage losses may be evaluated by making use of the capability of the computer program to solve partially saturated flow problems in heterogeneous media. Figure 6 shows the effects of an earth lining on the equipotentials and streamlines. The canal lining is 1 foot thick on the bottom, and 0.7 of a foot thick on the sides and has a permeability at a unit gradient of 0.163 ft.^{3}/ft.^{2}/day. These thicknesses were governed by an arbitrary choice of grid spacing during the calculations. The seepage loss in this case was reduced to 16.8 ft.^{3}/day/linear ft. of canal. The patterns of equipotentials and streamlines do not differ greatly from the unlined case. Comparison of figures 4 and 7 illustrates that the major difference in the system is in the moisture content pattern.

The consideration of two other earth-lined canals with identical geometry and less permeable linings permits comparison of seepage losses. Figure 8 shows this effect with the curved line representing the more realistic partially saturated flow system. The saturated flow from Muskat, where $\underline{K}/\underline{K}_o = 1$, is shown normalized with the true quantity of flow from the partially saturated system. The shape of the curve in figure 8 is particularly significant since a large reduction in permeability is required to achieve a moderate decrease in seepage rate. For example, if the saturated permeability of the lining is one-tenth that of the base soil, the seepage reduction factor is only 0.43. It is

Figure 6.

TWO DIMENSIONAL FLOW PATTERN BELOW A 15 FOOT EARTH LINED CANAL

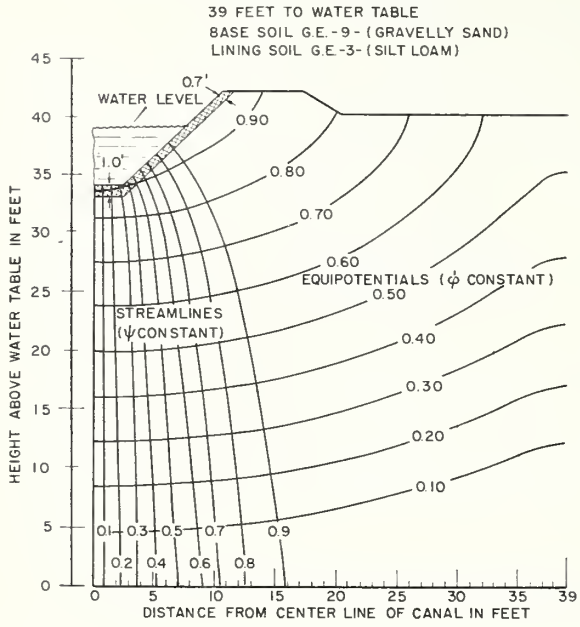


Figure 7.

MOISTURE CONTENT PATTERN BELOW A 15 FOOT EARTH LINED CANAL

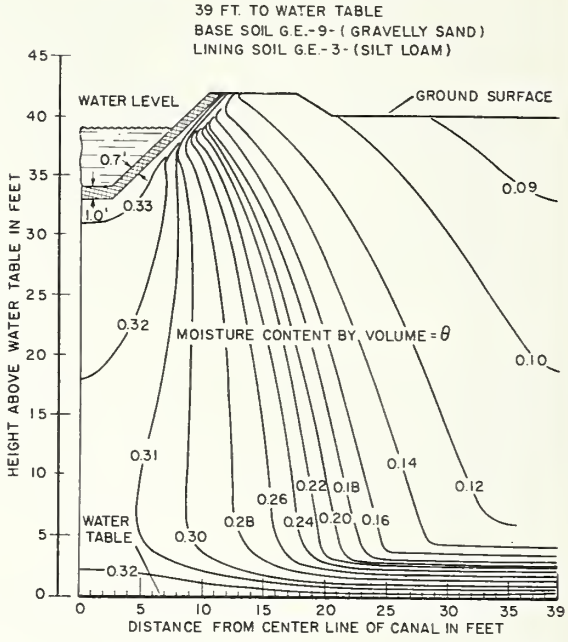
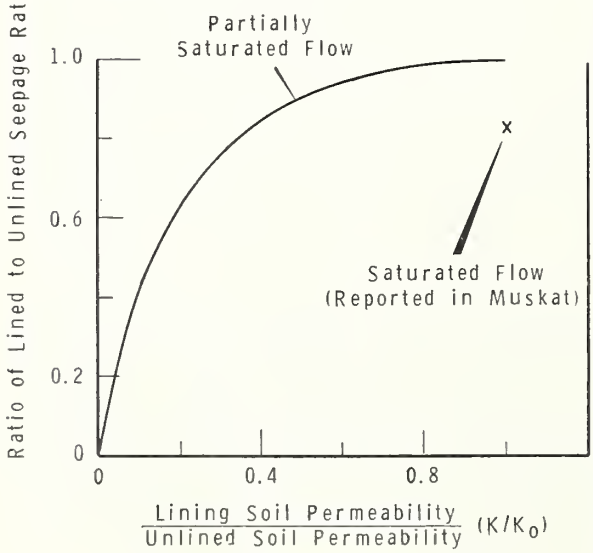


Figure 8.

EFFECT OF EARTH LINING PERMEABILITY ON CANAL SEEPAGE
(Depth to Water 39 Feet)

Canal Dimensions
Bottom Width 5 ft.
Water Depth 5 ft.
Side Slope 1:1
Soil G.E.-9
K_0 0.789 $ft.^3/ft.^2$ Day $\frac{ft.}{ft.}$
Q_0 26.1 $ft.^3$/Day/Lin. ft.)


Partially Saturated Flow
Saturated Flow (Reported in Muskat)
Ratio of Lined to Unlined Seepage Rate (Q/Q0)
Lining Soil Permeability / Unlined Soil Permeability (K/K0)

emphasized that this result is for a particular canal geometry, combination of soil types and depth to water. In general, the resulting curve for coarser textured soils and greater water table depths would be expected to lie to the left of the partially saturated curve in figure 8. For shallower water table depths and finer texture soils, the curve should lie between that curve and a diagonal line extending from the origin to coordinates (1,1). The latter straight line represents the assumed saturated flow condition. It is apparent that an evaluation of the effectiveness of earth linings in the reduction of seepage losses is markedly dependent upon the soil characteristics and boundary conditions of the flow system when the analysis is based on partially saturated flow conditions.

The influence of soil heterogeneity on seepage from a canal is important since all practical flow systems are to some degree heterogeneous. Figure 9 shows a heterogeneous pattern consisting of three soils. Several distortions caused by the silt loam layer occur in the equipotentials and streamlines. The moisture content pattern for this system (fig. 10) differs appreciably from the

Figure 9.

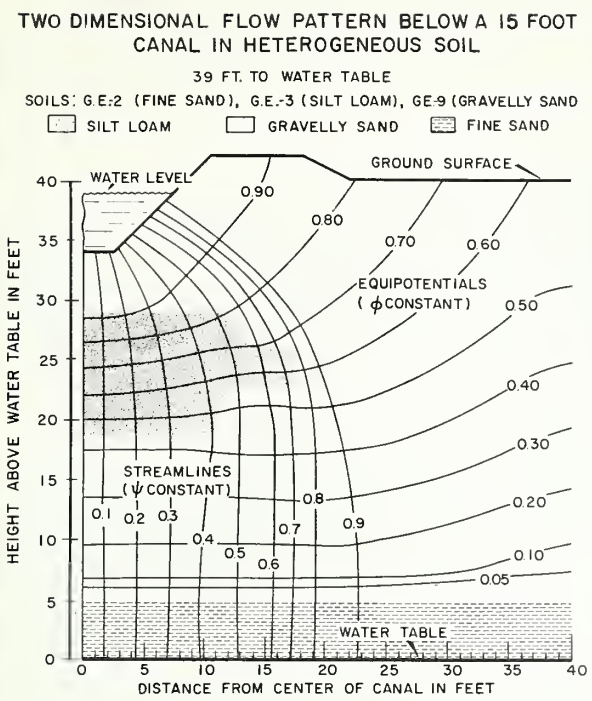
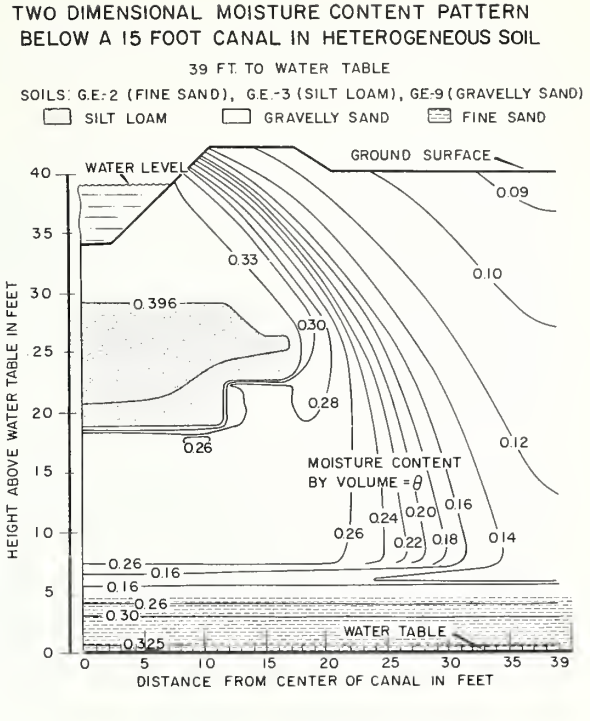

TWO DIMENSIONAL FLOW PATTERN BELOW A 15 FOOT CANAL IN HETEROGENEOUS SOIL
39 FT. TO WATER TABLE
SOILS: G.E.-2 (FINE SAND), G.E.-3 (SILT LOAM), GE-9 (GRAVELLY SAND
SILT LOAM
GRAVELLY SAND
FINE SAND
WATER LEVEL
GROUND SURFACE
EQUIPOTENTIALS
(ϕ CONSTANT)
STREAMLINES
(ψ CONSTANT)
WATER TABLE
HEIGHT ABOVE WATER TABLE IN FEET
DISTANCE FROM CENTER OF CANAL IN FEET

Figure 10.


TWO DIMENSIONAL MOISTURE CONTENT PATTERN BELOW A 15 FOOT CANAL IN HETEROGENEOUS SOIL
39 FT. TO WATER TABLE
SOILS: G.E.-2 (FINE SAND), G.E.-3 (SILT LOAM), GE-9 (GRAVELLY SAND)
SILT LOAM
GRAVELLY SAND
FINE SAND
WATER LEVEL
GROUND SURFACE
MOISTURE CONTENT BY VOLUME = θ
WATER TABLE
HEIGHT ABOVE WATER TABLE IN FEET
DISTANCE FROM CENTER OF CANAL IN FEET

homogeneous case (fig. 4). The lens of silt loam is saturated at a volume fraction moisture content of 0.396. The seepage loss in this case was 14.5 ft.^{3}/day/linear ft. of canal.

SUMMARY

A computer program capable of solving problems relating to multidimensional, partially saturated flow in heterogeneous soils has been briefly described and used to examine rates of seepage loss from canals. The losses based on a partially saturated flow analysis were found to be more than that predicted by the traditional saturated analysis. Depth to the water table was shown to have an effect on seepage rate. Under partially saturated flow conditions the seepage rate was found to decrease significantly with increased water table depth.

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EFFECT OF WATER QUALITY ON SEEPAGE THROUGH SOIL[1]

C. A. Bower[2]

Owing to the presence of excess negative electrical charges at their surfaces, soil particles adsorb and retain cations such as calcium, magnesium, and sodium. While the adsorbed cations are combined chemically with the soil particles, they may exchange with other cations in the soil solution. Each soil has a reasonably definite capacity to adsorb and exchange cations, and the percentages of this capacity which are satisfied by the various cations are referred to as exchangeable-cation-percentages; e.g., exchangeable-sodium-percentage (ESP).

Because cations adsorbed on soil particles can interchange freely with adjacent cations in the soil solution, the proportions of the various adsorbed cations (exchangeable-cation-percentages) are related to their concentrations in the soil solution. Calcium and magnesium are the principal cations found in the soil solution and on the particles of normal soils in arid regions. When waters containing a high proportion of sodium are allowed to flow through normal soils, sodium in the water replaces part of the originally adsorbed calcium and magnesium until an equilibrium is attained between the cations in the water and those adsorbed on the soil. Conversely, if water containing a high proportion of calcium plus magnesium flows through soil having high exchangeable sodium, replacement of the exchangeable sodium will occur. Knowledge of the relation between dissolved and adsorbed cations thus permits estimation of the ESP which will result when the water attains equilibrium with the soil.

Owing to the fact that calcium and magnesium are more strongly adsorbed than sodium and for other reasons, the proportions of these cations present in the adsorbed form are not the same as the proportions in the soil solution. The ratio of adsorbed sodium to adsorbed calcium plus magnesium is related in a linear fashion to an adjusted ratio of sodium to calcium plus magnesium in the soil solution termed the sodium-adsorption-ratio (SAR); viz: $SAR = Na/\sqrt{(Ca + Mg)/2}$, where the concentrations are expressed in milliequivalents per liter. Neglecting the usually low amounts of dissolved and adsorbed potassium than occur in soils, the ratio can be related to the ESP by a simple calculation.

For both saturated and unsaturated flow, the permeability of soils in arid regions is markedly influenced by the composition of the exchangeable cations and the total salt concentration of the soil solution. Adsorbed calcium and magnesium have a favorable effect on permeability whereas the effect of adsorbed sodium on permeability is quite adverse. Thus, increases in the ESP of the soil result in decreases in permeability. An increase in the total salt concentration of the soil solution tends to increase the permeability of soils regardless of their adsorbed cation composition, but the effect is quite marked if the ESP of the soil is appreciable. The permeability and ESP of the soil and the total salt concentration of the soil solution are, therefore, interrelated. If sufficient water has flowed through the soil to achieve equilibrium between the cations in the water and those adsorbed on the soil, then there is also an interrelation between the permeability of the soil and the total salt concentration and SAR of the water, since the salt concentration and SAR determine the ESP of the soil.

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[1]Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2]Director, U.S. Salinity Laboratory, Riverside, Calif.

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APPLICATION OF SEEPAGE METERS[1]

Herman Bouwer[2]

INTRODUCTION

Seepage meters are in principle suitable devices for measuring local seepage rates in canals or ponds. However, erratic results have sometimes been reported, and the need for refinements in equipment and techniques has been recognized. This paper reports on studies that were performed to identify and eliminate, as much as possible, sources of error. In the course of these studies, it became apparent that seepage meters not only can be used to measure seepage but also to evaluate hydraulic conductivity of bottom material and seepage gradients.

Seepage meters are cup- or bell-type devices. A common diameter is approximately 1 foot. The cylinder part, which is approximately 8 inches high, is pushed a small distance into the canal bottom. If the pressure head inside the seepage meter equals that in the canal, the outflow from the seepage meter is a measure of the seepage of the portion of the bottom that is enclosed by the seepage meter. Thus, the principle of measuring seepage with seepage meters is to maintain a pressure inside the seepage meter that is equal to that outside the seepage meter and to measure, at the same time, the outflow from the seepage meter. This, however, is easier said than done.

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[1]Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture.

[2]Research hydraulic engineer, U.S. Water Conservation Laboratory, Tempe, Ariz.

ANALYSIS OF ERRORS

There are at least three possible sources of error in measuring seepage with seepage meters. One may be in the disturbance of the bottom material when the meter is installed. Errors also may be introduced if, when the outflow from the seepage meter is measured, the pressure head inside the meter is not exactly equal to that due to the free water surface, which is generally taken as a reflection of the pressure head around the meter. A third source of error may be due to the pressure distortion around the meter in flowing water. At the upstream face of the meter, pressure increases due to stagnation can be expected. At the downstream face of the meter, decreased pressure heads are caused by a tendency for separation. The average head environment of the meter in flowing water may not be equal to that corresponding to the free water surface head, so that a seepage measurement based on maintaining the free water surface head inside the meter will be in error.

Errors due to disturbance of the bottom material may be reduced by minimizing the depth of penetration of the seepage meter into the canal bottom. In this case, however, it is necessary to prevent any surges inside the meter while the meter is pushed into the canal bottom. The surges may cause blowouts or imperfect contact between the seepage meter and the canal bottom material. To minimize pressure surges inside the seepage meter as it is forced into the canal bottom, a meter with a completely removable lid is recommended. Prior to installation, the lid is removed from the meter. After the desired depth of penetration has been reached, the lid is replaced.

Errors produced when the pressure head inside the seepage meter was not exactly equal to that outside the seepage meter during measurement of outflow from the meter were analyzed by resistance network analog. The analyses indicated that when seepage gradients are low, a small head difference of one-half inch or so could cause errors in the measured seepage that approached the magnitude of the seepage itself. For high seepage gradients, such pressure differences cause only a small error in the measurement. In general, it is advisable to base the seepage measurement on a pressure head inside the seepage meter that is as close as possible to that outside the seepage meter.

Pressure distortion around the seepage meter was analyzed in a hydraulic flume. The results indicated that the average head environment of a seepage meter in flowing water was a fraction of an inch below that corresponding to the free water surface. Again, the error caused by such a difference is only significant in case of low seepage gradients. Graphs and tables were prepared and published (3),[3] from which correction factors to be applied to the free water surface to obtain the proper reference level for seepage measurement in flowing water can be determined.

FIELD TECHNIQUES

Because of the difficulty in maintaining a head inside the seepage meter that is exactly equal to the head in the canal, a falling-head technique, whereby the seepage meter is connected to a falling-level reservoir (fig. 1), is used. Before the seepage is measured, the water level in this reservoir is raised an inch or so above the water surface in the canal. The subsequent fall of the water level in the reservoir is measured by means of a vacuum inverted U-tube manometer which is placed on the canal bank. One leg of the manometer is connected to the seepage meter interior, the other to the free water in the canal. The water level in the manometer tube connected to the seepage meter will fall,

[3] Underscored numbers in parentheses refer to Literature Cited at end of this paper.

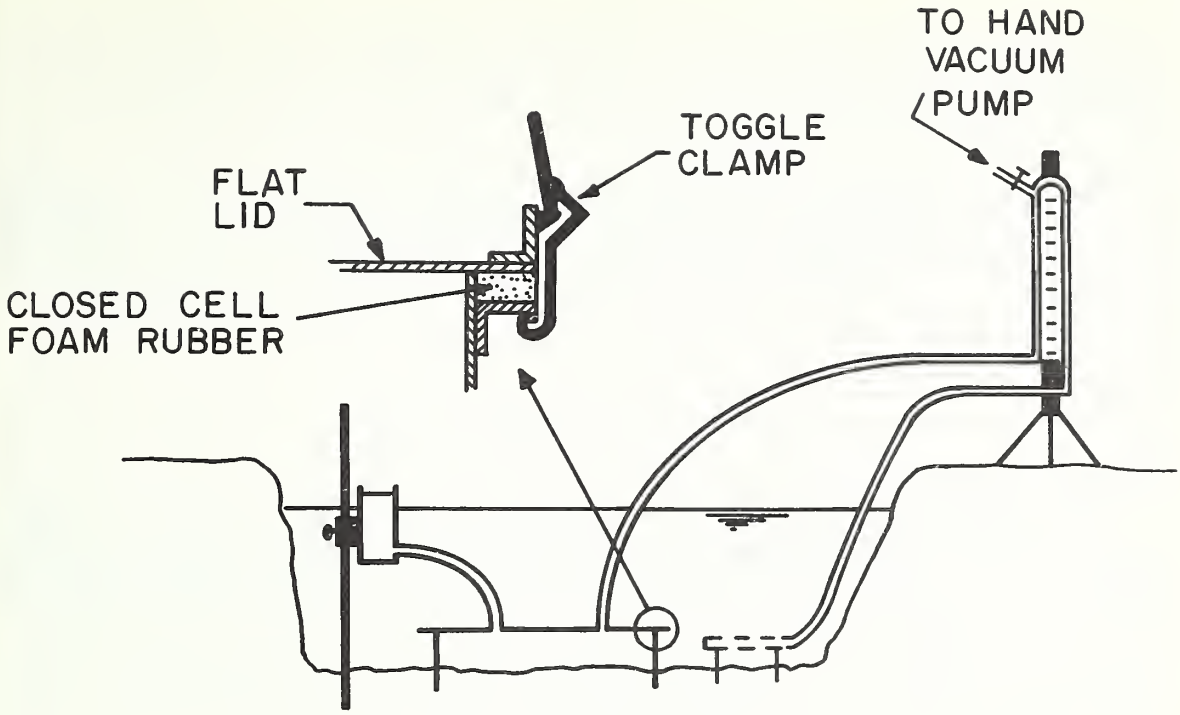

Figure 1.--Sketch of seepage meter equipment in canal.

whereas that in the tube connected to the free water surface in the canal will rise. At any time, however, the difference between the water levels in the manometer tubes is equal to the pressure difference between the seepage meter and the outside canal, even if the water level in the canal is fluctuating during the time of the measurements.

The field measurements consist of taking time and water-level readings of the water levels in the manometer. From these data, seepage can be calculated graphically or analytically with a falling-head equation. The graphical procedure is the easiest. It consists of plotting the manometer and time readings on graph paper. At the point of intersection of the two resulting curves, the pressure inside the meter equals that outside the canal, and the angle between the curves at their point of intersection can be converted into the seepage from the meter.

If the water surface in the falling-level reservoir is permitted to fall sufficiently long, it will reach an equilibrium position whereby the original outflow from the seepage meter due to seepage is compensated by an inflow component into the seepage meter caused by a lower pressure in the meter than in the canal. This condition has been called the balanced-flow condition. The corresponding pressure difference between the meter and the canal is the balanced-flow differential head.

The balanced-flow differential head, which can be measured rapidly in the field by closing the valve between the seepage meter and the falling-level reservoir, is an indication of the seepage gradient in the bottom material below the seepage meter. Since the seepage and the balanced-flow differential head are known, the hydraulic conductivity of the bottom material can be calculated through the aid of dimensionless flow factors which were evaluated with the resistance network analog. Graphs from which those flow factors can be determined for a particular seepage meter installation have been prepared by

Bouwer and Rice (4). The calculation of the hydraulic conductivity of canal bottom material is applicable if the bottom is uniform to rather large depths or if the bottom material is underlain by soil of much higher or much lower hydraulic conductivity. The depth of the material of different conductivity may vary.

Dividing the seepage by the hydraulic conductivity of the bottom material yields the seepage gradient, which can be a clue as to the conditions under which the seepage takes place. Knowledge of seepage gradients may enable one to predict how seepage will react to changes in water depth in the canal or in water table positions in the soil adjacent to the canal.

NATURAL OR ARTIFICIAL LININGS

Thin, slowly permeable, natural or artificial layers may exist at, or near, the surface of canal bottoms. If a seepage meter is installed, such a thin layer may be completely penetrated by the meter. In that case, the meter behaves as a falling-head permeameter, and the conductivity characteristics of the slowly permeable layer can be evaluated in terms of hydraulic impedance. This term is defined as the thickness of the slowly permeable layer divided by its conductivity. The dimension of hydraulic impedance is time, and it can be construed as the length of time required for a unit depth of seepage to take place under a unit head difference across the slowly permeable layer.

The combination seepage meter and slowly permeable layer can be considered as a very large tensiometer, whereby the meter represents the tensiometer cup and the slowly permeable layer the ceramic plate of the tensiometer. Thus, the equilibrium level inside the meter, after the valve to the falling-level reservoir is closed, can be used to measure the pressure or tension immediately below the slowly permeable layer. Laboratory studies were performed in a large tank, whereby excellent agreement was obtained between the impedence, seepage, and pressure below the slowly permeable layer as measured with the seepage meter and the results obtained from direct measurements.

For actual installations, seepage meter readings can be taken at various depths of penetration of the meter, and a plot of the balanced-flow differential head against depth of penetration of the seepage meter can be constructed. Such a plot yields information regarding the location and thickness of the slowly permeable layer and also of the pressure conditions immediately below the layer. Application of this technique in a golf course lagoon treated with a commercial sealant showed that most of the sealing action occurred between 1 and 2 inches below the canal bottom. The pressures below the seal were slightly negative; i.e., approximately -4 inches of water.

Another possible use for the seepage meter is in the evaluation of field performance of certain artificial soil sealants. The seepage meter is placed in the canal or pond to be treated, and the sealant is applied through the seepage meter. From the equivalent impedances before and after the application of the sealant, the impedance of the slowly permeable layer formed by the sealant can be computed. Repetition of this test at different depths of penetration of the seepage meter will yield information as to the depth and location of the sealed layer thus formed. This method may be a valuable step between laboratory studies and large field trials in evaluating the performance of water-borne sealants.

APPLICATION OF SEEPAGE METERS

Seepage meters may be used to locate leaky sections of canals and to measure distribution of seepage inside canals. They may also be used to check results obtained from logging techniques, such as the resistivity method and others. The possibility of measuring seepage gradients with seepage meters may be of value in ground-water recharge work to determine, for instance, if a decline in recharge is due to clogging of the spreading basin or to the rising of a ground-water mound below the basin. In the first case, seepage or recharge gradients will be high; in the second case, the gradients will be low.

An interesting aspect of the conductivity measurement of bottom material with seepage meters has been the development of a double-tube method for measuring hydraulic conductivity in place in the absence of a water table. Instead of a seepage meter in a canal, there is one concentric tube inside another one. The inner tube acts as the seepage meter, whereas the surrounding water in the outer tube acts as the water in the canal. If certain falling-water level measurements are made in the inner and the outer tube, the conductivity of the soil below the tubes in the auger hole can be evaluated (1, 2).

For a detailed description of the use of seepage meters in seepage and other investigations, reference is made to a paper prepared by Bouwer and Rice (4).

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DISCUSSION

Tuesday afternoon - Second Session

Subsequent to the presentation of the paper by A. E. Reisenauer, errors in the quantities of flow were noted. The text of the paper has been corrected and revised accordingly.

Helen Peters:

I would like to ask Mr. Resienauer some questions about his computer program. One, is the hydraulic conductivity, K, treated with the same value in all three directions, or can you apply to the computer program variable permeabilities in the horizontal and vertical directions?

Reisenauer:

The answer to that is: It is considered to have the same permeability in all directions. We haven't included a way of solving the anisotropic case in this computer program. This could be done, but as yet there is no method of in-place measurement of the necessary parameters.

Peters:

Another question is: Have you done any work with the nonstatic, the dynamic situation, or is your solution entirely a static situation.

Reisenauer:

I assume by nonstatic you mean transient flow systems where the velocity changes with time. This computer program considers only steady-state flow, where the velocity is independent of time. We have written a transient-flow program that will solve some types of problems, but have had limited success with it, mainly due to our inability to control instabilities in the computer program. We are experimenting along these lines. We are pursuing this further and hope to be able to master this in about a year and a half.

SESSION 3.--CHAIRMAN, DELL G. SHOCKLEY

INFLUENCE OF DITCH LININGS ON SEEPAGE LOSSES FROM FARM DITCHES IN SOUTHWESTERN NEW MEXICO[1]

Eldon G. Hanson[2]

The influence of ditch linings in southern New Mexico on reducing seepage losses and ditch operation and maintenance expenses is being determined in a joint research project with the Agricultural Engineering Department and the Agricultural Economics Department of the New Mexico Agricultural Experiment Station. The research program will concentrate on farm ditches which total approximately 1,400 miles in New Mexico and west Texas under the Elephant Butte Dam.

To determine savings of water and operation and maintenance costs, an intensive sampling throughout the area including 16 case farms is being made pertaining to:

1. Sizes of ditches, lined and unlined, and proportion of ditches that have been lined.
2. The proportion of ditches that are used as conveyance ditches and head ditches and the relative time of duration that each type of ditch is used throughout the irrigation season.
3. Savings in time, materials, and labor with lined ditches as compared to unlined.
4. Reduction of seepage losses from lined ditches by types of linings and turnouts as compared to unlined ditches.

This report presents the results of some of the seepage measurements which have been made during the preliminary work on this project. All of the seepage measurements have been made with water stage records placed in dammed sections of farm ditches. Tests on lined ditches were made with: (1) turnouts closed and sealed against leakage using plastic cement; and (2) turnouts closed without sealing according to usual farm practice. To seal turnouts for tests in earth ditches, moist soil was tamped in front of the turnouts. Figure 1 presents the results of seepage measurements on some of the ditches from three farms.

In figure 1, A, for farm A, curves 1 and 1A pertain to two adjacent sections of the same ditch. Curve 1 represents rate of loss in a section that had no turnouts and curve 1A shows the loss measured in a section with turnouts closed but not sealed. The loss from the section with turnouts leaking is approximately 35 times greater than that from the section without turnouts. Curves 2 and 3 compare losses from two ditches located near each other on another section of the same farm. Curve 2 for a concrete-lined ditch with turnouts unsealed shows the rate of water loss to be about as great as that from the nearby earth ditch indicated by curve 3. The curve for the earth ditch shows only seepage through the soil since the water surface was not sufficiently high to leak through the turnouts in this test.

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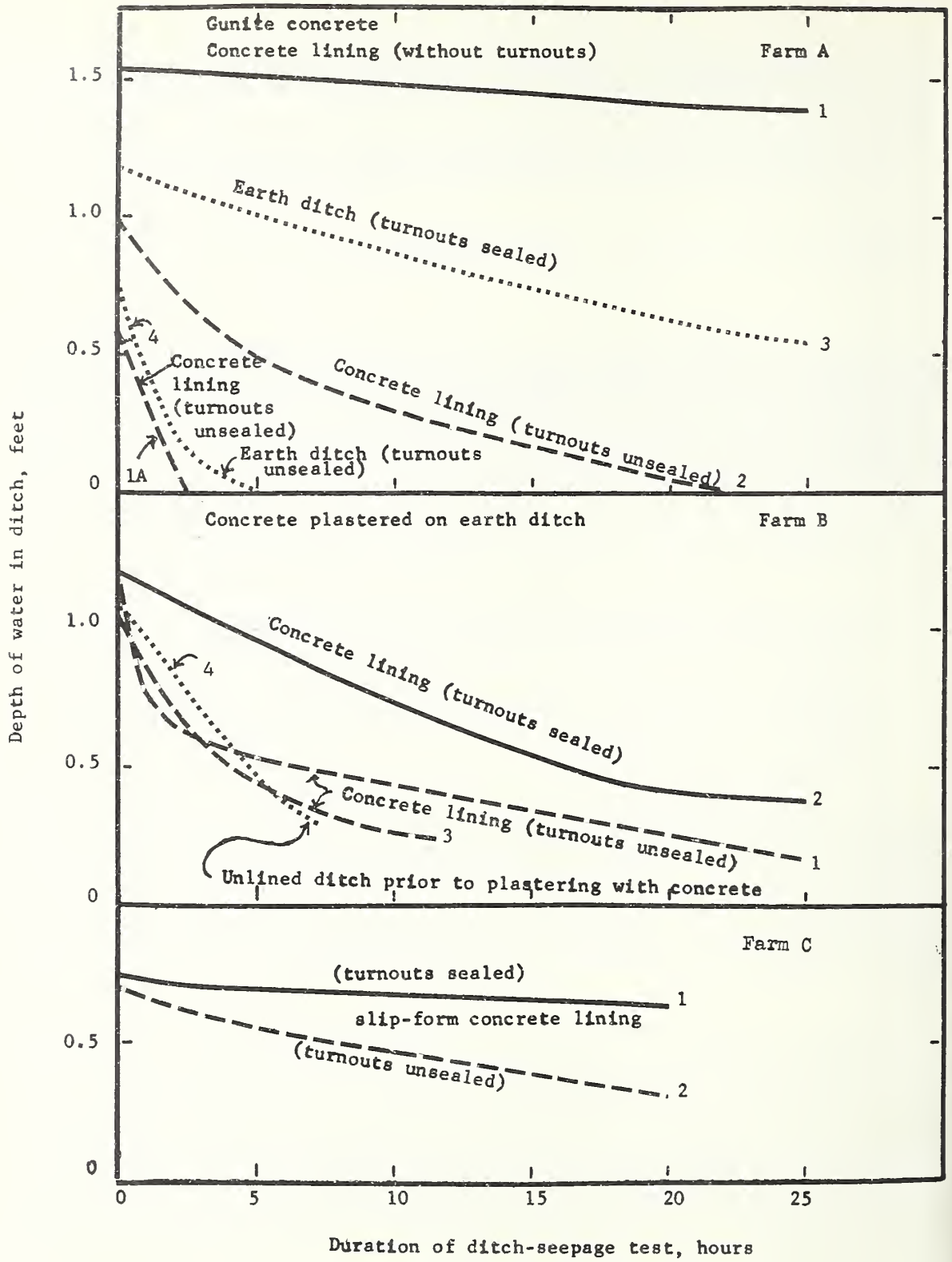
[1] Contribution from the New Mexico Agricultural Experiment Station, New Mexico State University, University Park, N. Mex.

[2] Head, Agricultural Engineering Department.


Gunite concrete
Concrete lining (without turnouts)
Farm A
1.5
1
Earth ditch (turnouts sealed)
1.0
3
4
0.5
Concrete lining (turnouts unsealed)
Concrete lining (turnouts unsealed)
2
1A
Earth ditch (turnouts unsealed)
0
Concrete plastered on earth ditch
Farm B
1.0
Concrete lining (turnouts sealed)
4
2
0.5
Concrete lining (turnouts unsealed)
1
3
Unlined ditch prior to plastering with concrete
0
Farm C
(turnouts sealed)
1
slip-form concrete lining
0.5
(turnouts unsealed)
2
0
0
5
10
15
20
25
Depth of water in ditch, feet
Duration of ditch-seepage test, hours

Curve 4 shows the seepage rate in a nearby earth ditch from which water was leaking through unsealed turnouts. A comparison of curves 1A and 4 shows a rate of water loss from the lined ditch to be greater than the loss from the unlined ditch.

In figure 1, B, for farm B, curves 1, 2, and 3 represent three adjacent sections of a concrete-lined ditch which has been plastered with concrete by farm labor. The center section (curve 2) had turnouts sealed and the sections on either side (curves 1 and 3) had turnouts unsealed. The losses from the unsealed sections were more than four times as great as those from the sealed section. Curve 4 represents seepage losses from the ditch as it existed without lining approximately 1 year before the concrete lining was installed. Soil had been packed in front of each turnout when this test was made in the earth ditch.

Figure 1, C, shows losses from a concrete-lined ditch which was constructed with the slip-form method on the agronomy farm of the New Mexico Agricultural Experiment Station. The seepage loss from the section of the ditch with turnouts unsealed (curve 2) is more than three times as great as the loss from the section with the turnouts sealed (curve 1). The slope of curve 1 on this figure is approximately the same as that of curve 1 in figure 1 where the ditch had no turnouts. This slope represents approximately the minimum rate of loss that has been measured to date on all farms.

A summary of all the seepage measurements to date shows that approximately 88 percent of the water lost from lined ditches occurs by leakage through closed turnouts. These data emphasize a great need for improved low-cost turnouts in order that the potential water savings by lined ditches may be realized.

A LOWER COST CANAL LINING PROGRAM[1]

Richard J. Willson[2]

The timely theme and important purpose of this symposium on seepage, in a broad sense, is conservation of water and land. The individual topics have been or will be devoted to: Seepage problems on the farm or on an irrigation project; the study of seepage losses from natural channels or manmade canals and reservoirs; and methods of evaluating and correcting seepage problems. Overall, the topics are concerned with some phase of the all-important conservation of water resources and agricultural land.

The Lower Cost Canal Lining Program, initiated in 1946 by the Bureau of Reclamation, is but one of many studies presently underway in the United States toward conservation of water and land and is a program that has contributed to such conservation in a particular field--the water serving the irrigable land on our irrigation projects in the 17 Western States.

The topic assigned me for discussion at this symposium is the Bureau of Reclamation Lower Cost Canal Lining Program. It can be briefly reviewed by answering three questions: Why did the Bureau of Reclamation undertake a Lower Cost Canal Lining Program? What has been accomplished in the intervening 17 years since the program was initiated in 1946? What more can be accomplished?

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[1] Contribution from the Bureau of Reclamation, U. S. Department of the Interior.

[2] Civil engineer, Office of Chief Engineer, Denver, Colo.

PURPOSE OF PROGRAM

It has been pointed out by the Senate Select Committee in its consideration of water resources that, "In any irrigation project only a portion of the total available water supply is put to beneficial use in growing crops . . . actual consumptive use on a project may be but one-half or less of the available supply" (1).[3] The saving of water by the control of seepage has been the primary objective of the Bureau's Lower Cost Canal Lining Program and is one phase of the water conservation problem considered by the Senate Committee. Specifically, the Bureau's program was authorized by Congress to study means of saving water by the reduction of seepage losses from canals and laterals in the Bureau's carriage and distribution systems at less cost, and by so doing, as a second objective, alleviate seepage problems on irrigable project land.

Water losses from canals and laterals through seepage have been satisfactorily reduced through the installation of relatively impervious linings, by special treatment of the canal section, or by the use of closed conduits. However, the cost of accepted and dependable methods of seepage control in general use at the time the Lower Cost Canal Lining Program was initiated was prohibitive for many irrigation systems. Costs increased considerably along with all construction activities following World War II as the need for water conservation in the irrigation states also increased.

A considerable amount of individual, uncoordinated effort to reduce the cost of lining canals has been expended for short periods in the past. However, such effort was usually short-lived and resulted in very limited progress. In recognition of the urgent need for an organized and continuing effort, the Bureau of Reclamation inaugurated its Lower Cost Canal Lining Program. The program called for laboratory and office research, surveys of existing installations, seepage determinations, field experiments, and equipment developments aimed at reducing the cost of seepage control.

An interim report issued in 1946 outlined the broad scope of the Lower Cost Canal Lining Program and presented information available at that time on various types of existing linings and their service records. The aim of the program and some of the problems involved were brought to the attention of the engineering profession in February 1947 (5), and an appeal was made to contractors and equipment manufacturers for their assistance in developing lower cost construction methods and greater mechanization of equipment. Progress during the first 2 years was covered in a general information report dated June 1948. In March 1949, a committee was formed to direct the program activities. Composed now of one member from each of the Bureau's seven regional offices and three from the Office of the Chief Engineer, the committee meets annually to review program accomplishments, to formulate fiscal year programs, and to arrange for funds to finance the work.

The scientific talent of several universities and colleges, the Soil Conservation Service, the Agricultural Research Service, the Geological Survey, and others has been enlisted on various phases of the laboratory and field investigations and studies. New development and condition surveys are reported in an exchange of quarterly reviews and in special reports as required, and an annual report of the program accomplishments and activities is prepared. In addition, the committee is responsible for the periodic publication of a summary of its laboratory and field investigations, its studies, and, as soon as adequate evaluation has been possible, recommendations for the construction of linings or adoption of testing and construction standards. Such a summary was first published in 1948 (3), and a second

[3] Underscored numbers in parentheses refer to Literature Cited at end of this paper.

in 1952 (4). A third publication, "Linings for Irrigation Canals--Including a Progress Report on the Lower Cost Canal Lining Program," is now in the hands of the printer and should be available for distribution within the next few months.

PRINCIPAL PROGRAM ACCOMPLISHMENTS

After 17 years of organized effort under the Lower Cost Canal Lining Program, improved and less costly construction procedures have been developed for use with older types of linings and other means of seepage control, and other materials have been adapted or developed for use as lining. Linings of some type have been placed in the canals and laterals of most all of the projects constructed by the Bureau of Reclamation in the 17 Western States. Our records do not include all linings placed in canals and laterals by project forces, nor do we have a complete record of the canal linings placed prior to 1947. However, we do know that about 3,000 miles of the more than 30,000 miles of canals and laterals constructed have been lined, table 1, since the Bureau of Reclamation started keeping records in 1947 with the beginning of the Lower Cost Canal Lining Program. More than 2,600 miles, or about 90 percent of the linings installed have been of the lower cost type.[4]

Evidence that both lower cost seepage control and significant savings have resulted from our efforts is found in construction records and from a comparison of construction

Table 1.--Quantities of canal linings placed on Bureau projects

Type of lining	Square yards	Miles
Exposed linings		
Asphaltic concrete (hot- and cold-mixed)	276,000	42.0
Asphalt macadams	11,000	0.8
Asphalt surface membranes (prefabricated and constructed-in-place)	81,000	10.3
Other exposed asphalt linings (mortars, blocks, and slabs)	8,000	0.7
Portland cement concrete (unreinforced)	23,690,000	1,077.5
Portland cement concrete (reinforced)	9,738,000	419.9
Portland cement mortar (pneumatically applied)	1,915,000	167.6
Soil-cement	37,000	3.7
Other exposed linings (concrete blocks, plastic, and rubber surface membranes, etc.)	8,000	6.8
Buried membrane linings		
Asphalt, hot-applied	5,839,000	333.1
Asphalt, prefabricated	24,000	2.1
Bentonite	300,000	19.6
Plastic	22,000	1.2
Earth linings		
Thick compacted earth	12,152,000	566.2
Thin compacted earth	1,885,000	74.3
Loosely placed earth blankets	3,207,000	174.9
Bentonite-soil mixtures	108,000	6.1
Soil sealants (chemical, petro-chemical and sediment)	535,000	41.2
Miscellaneous		
Includes resurfacing of existing linings, their undersealing and grouting, and the construction of cast-in-place concrete pipe in lieu of lining	144,000	45.8
Total	59,980,000	2,993.8

[4] The total of 2,993.8 miles shown in table 1 includes 419.9 miles of reinforced concrete lining, which is not considered a lower cost type.

costs based upon bid prices as presented in table 2. Factors contributing to the achievement of the program objectives are discussed below:

Liberalization and Simplification of Requirements

The liberalization and simplification of lining specifications requirements with respect to line, grade, and finish of hard-surface linings such as those composed of portland cement concrete, have encouraged greater mechanization of placing equipment and resulted in lower construction costs and, therefore, lower bid prices. A general price increase has occurred, in line with other construction costs, but the comparison still holds.

Elimination of Reinforcement

Savings of 10 to 15 percent in total cost have resulted from the elimination of reinforcement steel in portland cement concrete linings. Except in specific instances where structural safety is imperative, reinforcement steel is not now used.

Standardization of Canal Shapes and Sizes

Standardizing canal shapes and sizes, within a base width of 2 to 6 feet, has aided in the standardization of slip form equipment, requiring less capital investment and thus contributing to lower construction costs. The standardization of canal design has also resulted in lower design costs and faster completion of engineering details.

Development of Subgrade Guided Slip Forms

The development of subgrade guided slip forms has made it economically feasible to line small canals and laterals with hard-surface-type linings which heretofore have been prohibitive in cost. These slip forms have now been adopted in the placement of asphaltic and portland cement concrete and mortar linings.

Improvement of Methods of Measuring Seepage Losses

Development of a lining material that would be sufficiently low in cost as to permit economical lining of all canals and laterals will probably never become possible. Economics normally dictate that lining efforts be concentrated on those waterways or sections of waterways that are the more permeable and hence can benefit more by the effort. Locating the more permeable reaches of a canal or lateral has always been a difficult problem, and in many instances linings have been placed in much longer reaches than necessary because it was not possible to isolate the more permeable sections. Significant savings in lining costs could be realized if it were possible to determine accurately the amount and source of seepage losses from unlined canals.

New devices and test procedures are under development for more economical location and measurement of seepage losses. The reliability and adaptability of these methods are being established by basic investigations and field tests.

Use of New Lining Materials and Construction Techniques

As shown in table 1, many materials have been tested and some have been adopted for general use in the control of seepage from canals and laterals. We have learned the importance of choosing the right kind of lining to meet specific conditions, we know what to

Table 2.--Construction costs of canal linings based on bid prices

Type of lining	Cost per square yard[1]
Exposed linings	
Asphalt concrete (hot, plant-mixed):	
2 inches thick	[2] $2.30
4 inches thick	[3] 4.90
Asphaltic mortar, pneumatically applied:	
1-1/2 to 2 inches thick	3.70
Asphalt macadam, 2 to 4 inches thick	[2] 2.15
Asphalt, prefabricated surface membrane, 1/2 inch thick	[2] 1.55
Portland cement concrete:	
2 inches thick, unreinforced	2.11
2-1/2 inches thick, unreinforced	2.41
3 inches thick, unreinforced	2.67
3-1/2 inches thick, unreinforced	3.73
4 inches thick, reinforced	[4] 8.11
4-1/2 inches thick, reinforced	[4] 9.49
Portland cement mortar, pneumatically applied:	
1-1/2 inches thick, unreinforced	[2] 1.97
1-1/2 inches thick, reinforced	[2] 2.12
2 inches thick, unreinforced	[2] 2.00
2 inches thick, reinforced	[3] 2.31
Precast portland cement concrete blocks	[3] 3.10
Plastic surface membranes, 17-1/2 mils thick	[5] 2.00
Soil cement:	
Plastic type, 3-1/2 to 4 inches thick	[3] 1.10
Standard type, 5 inches thick	1.60
Buried membrane linings	
Asphalt, hot-applied, with cover of:	
Earth and/or gravel	1.10
Macadam	[5] 2.00
Shotcrete	[5] 2.60
Asphaltic materials, prefabricated, with cover of:	
Earth and/or gravel	[5] 1.10
Macadam	[5] 2.00
Shotcrete	[5] 3.00
Bentonite, 1-1/2 to 2 inches thick	1.10
Plastic material with earth and/or gravel cover:	
Polyethylene:	
3 mils thick	[3] 1.90
10 mils thick	[3] 1.80
17-1/2 mils thick	[3] 2.10
Polyvinyl chloride:	
8 mils thick	[3] .67
10 mils thick	[3] .75
Nylon fabric, coated with neoprene, with earth and/or gravel cover	[3] 3.00
Earth linings	
Thick compacted earth, 12 inches or more in thickness	.84
Thin compacted earth, less than 12 inches in thickness	.57
Loose earth blankets:	
Without gravel cover	.20
With gravel cover	.30
Bentonite-soil mixtures:	
Premixed	[2] 1.30
Mixed-in-place	[2] .40
Sealants and stabilizers	
Waterborne type	[5] .25

[1] Costs are for contract construction over period 1956-60, except as noted.
[2] No representative cost data available for period 1956-60. Cost given is for work done prior to 1956, generally by project forces.
[3] Cost represents construction by project forces and may not include equipment depreciation costs.
[4] Cost based on short reaches in vicinity of structures.
[5] Cost is for experimental installations which reflect field research and study costs.

expect from a given lining, and we can fit construction of dependable lining to materials available for their construction. Some of the materials that have been used are discussed briefly below.

Asphalt

Asphalt was given early consideration in the canal lining studies and has been used in many ways, providing an early breakthrough in the reduction of lining costs. It is a comparatively inexpensive material and is manufactured in many types, grades, and compositions. Numerous variations are possible in its application to the lining and waterproofing of canals, some at only a fraction of the cost of linings constructed of other materials. Steam-refined asphalt-cement is used as the binder for asphaltic concrete or is further processed to form cutbacks, emulsions, or airblown asphalts for a variety of uses. The addition of certain catalytic agents during the airblown process imparts desirable characteristics to asphalt for use as a membrane. Buried membranes of asphalt have been developed which may be constructed of prefabricated strips or of hot, sprayed-in-place asphalt-cement covered with a protective blanket of earth or gravel at a cost of less than one-half that of most hard-surface linings.

Bentonite

Bentonite, a montmorillonite-type clay, has been used for many years as a lining material. Under certain conditions, it has been very satisfactory as a low-cost lining material. Specifications have been written to cover the characteristics it is believed bentonite must possess to be most suitable for the work and for the method of construction, including the necessary earth and gravel cover that must be provided to protect the bentonite when utilized for a membrane-type lining. The original cost of linings of this type in areas where deposits of a good quality bentonite are nearby has been competitive with buried asphaltic membranes.

Plastics and Synthetic Rubbers

Plastic film and synthetic rubber suitable for canal lining have been and are still under test in the field and in laboratories by the Bureau of Reclamation, by the Agricultural Research Service, and by others. The first large-scale installation was made in 1961. Until recently, the cost of suitable plastics and synthetic rubber discouraged their general use as low-cost canal lining materials; but with the lowering of production costs through improved manufacturing and fabrication practices, the plastics have become competitive costwise with other types of membrane materials. While the very thin plastics and synthetic rubber films have not been successful when used as exposed linings, they are proving very effective and durable as buried membrane linings as long as the protective cover remains intact. Several recent full-scale installations of this kind utilizing plastic sheets have been made on Bureau projects. Thicker and, of course, more costly synthetic rubber sheeting is performing well as an exposed lining material.

Compacted Earth

Use is being made of relatively thick (2- to 3-foot) linings of compacted earth. Where suitable earth material is available with a minimum of haul and the job is large enough to fully utilize mechanized equipment, this type of lining has proved to be one of the lowest in cost for dependable seepage control.

Chemicals

The use of chemicals and related materials to reduce the permeability and increase the stability of soils traversed by canals is being studied extensively in cooperation with industry. Resins, lime, portland cement, asphalt, and petrochemicals have the ability to provide impermeability and stability to soils, but many are either expensive or toxic. However, based on laboratory and field tests, some of these chemicals appear promising and a concerted effort is being made to further their development. The use of waterborne chemical sealants appears to offer possibility as an economical means of reducing seepage from waterways. This method, whereby the sealing material is transported by the flowing water, would be particularly adaptable in the southwest where canals are in constant use. A number of test installations of waterborne chemical sealants have been made, and the immediate reduction in seepage losses has averaged about 67 percent; the benefits, however, apparently decrease with time in most instances. Full evaluation has not been completed, but it has been demonstrated that water can be saved with such sealants at reasonable cost. Further studies directed toward the development of more effective, durable, and economical sealants are underway.

Unreinforced, Cast-in-Place, Concrete Pipe

The use of unreinforced, cast-in-place, concrete pipe in lieu of linings has received considerable attention in the last 5 or 6 years. Two types of pipe have been constructed in a continuous-placement operation. One type is placed in two parts; the invert is placed first, which is followed by the remainder of the pipe. The other type is constructed monolithically in a single operation by patented machines designed to travel in a previously excavated trench. Using one of these methods, pipe having a diameter of 24 to 48 inches has been constructed for little more than it would have cost to construct concrete lining for a canal of equal capacity.

Portland Cement Concrete and Mortar

Portland cement concrete, particularly, and, to a lesser extent, portland cement mortar still occupy an important place in our canal lining program because of their known durability and serviceability. With simplification of specifications requirements, the elimination of reinforcement, the standardization of canal shapes and sizes, and the development of subgrade guided slip forms, concrete linings can be classed as lower cost types.

Some Direct Program Benefits

It is difficult to evaluate all the benefits derived from canal linings, but experiences with lining installations on the North Platte project in Wyoming and Nebraska, the Huntley project in Montana, the Riverton project in Wyoming, and elsewhere are extremely gratifying. On one project, some 2,100 acres of cultivated land had become so waterlogged by seepage from canals and laterals that it had to be abandoned. Numerous open drains had been constructed, but these were not sufficiently effective since much good farmland continued to be waterlogged and many farmsteads, including the basements in farm homes, gradually became flooded. Linings placed in canals and many principal laterals have now reduced the seepage to the point that cropping is again possible in many of the fields. Some open drains are now being filled and the areas occupied by the drains returned to cultivation. Similar benefits can be claimed on most projects where linings have been placed.

Before linings were placed in canals of the Fort Sumner project in New Mexico, the limited capacity of one canal made it impossible to get sufficient water through unlined sandy channels to serve the project area.

On the Milk River project in Montana, the placement of a lining eliminated a serious seepage problem--one that was endangering a rail line by saturation of the subgrade to such a degree that the banks of the nearby river were caving and sloughing badly.

The development of new types of linings has increased the number of competitive materials, thereby stimulating a reduction of contract bid prices.

The above are but a very few of the tangible benefits achieved and the operating problems solved. Other benefits resulting from linings could be enumerated, but it will suffice to state that satisfactory lower cost lining and lining procedures have been developed, and that by the use of the lower cost type linings it has been possible to provide seepage control on waterways that never could have been accomplished with the more expensive linings in general use in the past. The cost would have been prohibitive.

FUTURE STUDIES

The planners, designers, constructors, and operators of canal systems have been provided with durable and lower cost linings developed through studies made over the past 17 years. But full use of even the low-cost-type linings now available will fall far short of meeting the ultimate objective of conserving all possible water and relieving all possible irrigable lands from waterlogging. The Bureau's continuing efforts in its Lower Cost Canal Lining Program will be directed toward:

1. The development of devices and test procedures for more economical location and measurement of seepage losses, so that lining requirements can be more precisely determined.
2. Simplified construction procedures to lower construction cost and improvement of specifications covering construction of known types of linings to assure better service.
3. Continued field evaluation to determine the service rendered by the various types of linings and other means of seepage control. It is only through service records and knowledge of the expected life of a lining that the actual cost can be determined.
4. New developments in the relatively low-cost asphalts, plastics, and related materials with a view to their possible application as linings. Many such new products are continuously being developed.
5. Continued search for improved sealants as the most economical means of providing more universal linings and watertight canals and laterals. The ideal sealant would be one which is low in cost, effective, durable, and easily applied; and of the several types, the chemical sealants appear to be most promising. The assistance of the world's technical and scientific investigators is being enlisted in exploratory investigations to develop improved waterborne chemical sealants. As a part of this effort, all known data on the subject have been assembled into a single reference volume that has been given wide circulation (2).

SUMMARY

New materials have been developed. New and more efficient techniques have been utilized for construction. Revised design standards are now used. More accurate and dependable means have been devised to locate seepage losses. All these factors have contributed to lower costs in lining our project waterways.

Through the lower cost linings we are saving water, reducing or preventing the loss of good irrigable land, and, indirectly, saving some drainage construction costs.

There is more that can be done toward water and land conservation. Chemicals offer promise as soil sealants, and we are continuing to evaluate these materials, as well as new conventional lining materials and other lining techniques. We also are taking a long look at the economy in operation and maintenance that can be provided by underground conduits, particularly in our smaller distribution systems.

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CONVEYANCE LOSSES IN IRRIGATION SYSTEMS AND MEASURES FOR CONTROL[1]

C. W. Lauritzen[2]

The invitation to present a paper on lower cost linings carried with it the suggestion that I might like to interject a little of my philosophy on the subject. This appealed to me and may well be of greater interest to you, since practically everthing I know on the subject of lower cost linings has been published and much of it by some of the distinguished people in the audience. Besides, as you know, it is much easier to deal in generalities.

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[1] Contribution from the Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture, in cooperation with the Utah Agricultural Experiment Station.

[2] Soil scientist, Soil and Water Conservation Research Division, Agricultural Research Service, U.S. Department of Agriculture, Logan, Utah.

It is only within the past 10 to 15 years that water has been recognized as a valuable resource and one that might well be the limiting factor in our national development. True, those of us in the west and other arid and semiarid regions of the world have long recognized the value of water, but only recently has it received national and international attention.

Because water is a valuable and limited resource, we must find ways to use the supply as efficiently as possible. Irrigated agriculture is one of the major users, and it is with irrigation water that we are primarily concerned in this symposium. In order to get water to the land, it must be conveyed; to get water into the soil where it can be used, it must be applied. Based on the best information available, it appears that about one-third of the water diverted for irrigation purposes is lost in conveyance, and that only one-half of what is left when applied is stored in the root zone. No great accuracy is claimed for these estimates.

The 1959 Census of Agriculture shows that 15.8 million of the 73 million acre-feet of water conveyed in the 17 Western States and Louisiana--or 21.7 percent--were lost in conveyance. This appears low in light of previous estimates. In 1948 the Bureau of Reclamation reported in a progress report that 37 percent of all the water diverted on 46 of its projects was lost in conveyance. Of this loss, 23 percent was attributed to seepage and 14 percent to operational waste. These figures do not take into consideration losses in farm ditches, which in the aggregate are substantial, even though they are not in operation continuously. Some of these losses undoubtedly appeared as return flow to provide water for the irrigation of lands in other areas. Losses of this order were also reported by the Salt River Water Users Association. They reported a loss of 36 percent in 1950. After spending 6 million dollars[3] over a period of 6 years on canal improvement, including lining, the loss was reported to be 32 percent--a decrease of 4 percent and a saving of 48,000 acre-feet of water.

Studies of Israelsen and others[4] and Tomlinson and Woodward[5] indicate that only about one-half of the water applied to land is stored in the root zone; the rest contributes to deep percolation and runoff. Again we must recognize that some deep percolation is desirable, and that a part of the tail water is put to use. Assuming the figure of one-third for conveyance losses and one-half for losses associated with application, only one-third of the water diverted contributes to crop production. There are those who would question these values, and it is entirely possible that they are too high, but I think we will all agree that a substantial part of the water we divert for irrigation purposes is wasted in conveyance and application. I think you would have to agree also that the opportunity for augmenting our water supply by controlling these losses is far greater than for all other opportunities combined. This I say without attempting to minimize the importance of soil-water-plant relationships, evapotranspiration, and other factors relative to water requirements for crop production.

The questions are: How do we accomplish the goal? What are the economic limitations? Where are we headed?

Technological developments in the past 20 years have been world shaking. New materials and techniques have been developed to the extent that our thinking has been revolutionized. For the most part, advancements have not been as great in irrigation as in

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[3] Anonymous. Arizona irrigation group seeks loan for expansion. Engin. News Rec. July 21, 1955.

[4] Israelsen, O. W., and others. Water application efficiencies in irrigation. Utah Agr. Expt. Sta. Bul. 311. 1944.

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some other fields. As mentioned, the key position of water has been recognized only whtin the past 10 years or less. We can expect even greater emphasis in the future. A part of the reason for the lag in agricultural advancements has been the unfavorable position of agriculture. The accomplishments of industry have been phenomenal, and I anticipate a closer alliance between industry and agriculture. Food and fiber will always be important, and surplus may not be the problem some day that it is today. When insufficiency becomes the governing concern, there will be many changes in our outlook.

Costs are relative, and as overall situations change we can expect more innovations in agriculture. Irrigation today is being recognized as offering one of the best opportunities for augmenting agricultural production. While agricultural surplus is a current problem in our country, this is not true the world over. In many countries the problem is to supply the population with the bare necessities of life. Increased production without some population control is futile. Nevertheless, it behooves us to make the best use we can of our resources, and water is not the least of these. With proper water management, extensive areas in the world could support populations far in excess of those now being supported.

I do not pretend to be an authority on sociological problems, but it seems to me it is a mistake for scientists to ignore the sociological impact of their contributions. It seems to me also that too often the scientist takes the position that his work is concerned only with the advancement of new knowledge, leaving it to the politicians, economists, and sociologists to determine what is best for mankind. Maybe this is as it should be, but I believe that the scientist has a broader obligation.

I was asked, as I have said, to discuss lower cost canal linings, and I do not intend to ignore the assignment completely. With canal and reservoir linings, as with most things, one gets about what he pays for. A corollary to this might be the saying, "You pay for something you need whether you buy it or not." This is especially true of linings. Before it can be determined if one lining is lower in cost than another--in fact, before it can be determined if it is adapted to the job under consideration--the performance which can be expected of linings must be known.

It is generally conceded that we line canals and reservoirs primarily to save water. It has been my observation, however, that canals and farm ditches are lined in many instances, not so much because of the opportunity they afford for saving water, but for other reasons--such as channel stabilization, reduced maintenance requirements, and reclamation of seeped-out areas. This could change. What then should be the function of a lining and what physical properties should a lining have, if it is to perform these functions?

I think we would all agree that the first requirement of a lining is that it be watertight, or reasonably so. At first, this may seem at variance with the statement that the primary reason for lining many canals is not to save water. A lining must also be reasonably stable through the range of operating conditions to which it will be subjected, or it will be destroyed. Assuming that these requirements are met, it then becomes a matter of relative cost--not first cost alone, but cost based on maintenance requirements and losses which may result from interrupted deliveries of water. The cost should be amortized over the expected service life of the lining. Reasonably good information on construction costs is available. These costs are summarized in table 1. Data on maintenance costs, however, are limited and difficult to assemble, since much of this work is done piecemeal without any attempt to separate the cost of lining maintenance from other operating expenses.

Table 1.--Cost of canal linings

Lining	Cost per square yard[1]			Relative cost index
	Material	Installation[2]	Total	
Concrete, 3-inch, unreinforced	$1.25	$1.75	$3.00	100
Shotcrete, 2-inch	[3] 1.90	.40	2.30	77
Earth, heavy compacted	--	1.00	1.00	58
Earth, heavy, compacted gravel topping	--	1.30	1.30	76
Bentonite, 27 lb./sq. yd.	.27	.73	1.00	58
Soil-cement, 4-inch	--	--	1.00	33
Asphaltic concrete	--	--	2.40	80
Prefabricated plank type	[4] 2.26	.60	2.86	93
Asphalt-coated jute	.85	.30	1.15	38
Buried asphaltic membrane, sprayed	[3] .33	.60	.93	54
Buried asphaltic membrane, prefabricated asphalt-coated jute	.85	.65	1.50	87
Polyethylene film, buried, 10 mil	[5] .23	.62	.85	50
Vinyl film, buried, 10 mil	[5] .53	.62	1.15	68
Butyl sheeting, 30 gage, buried	1.25	.62	1.87	109
Butyl sheeting, 60 gage, exposed	2.40	.10	2.50	83
Butyl sheeting, 60 gage, buried	2.40	.62	3.02	176

[1] Assuming earth and buried membrane linings will require 1 3/4 as much lined surface per unit length.
[2] Includes fine grading of subgrade and cover material where required (a uniform charge of $0.60 was made for preparing and covering all buried membrane linings).
[3] In place.
[4] Includes freight. At $0.60/cwt., f.o.b. cost $1.66/sq. yd.
[5] Altus and Tucumcari job.

Another factor not always considered involves the relative capacities of canals with hard-surface linings compared to the capacities of canals lined with other materials. Although the cost per unit area of lining is less for some of our so-called lower cost linings, the cost per unit capacity may not be much less, and hard-surface linings may actually be cheaper when service life and maintenance are considered. Where differences in elevations between the point of diversion and delivery make it possible to have stream velocities in excess of permissible velocities for alternate types of linings, they may be cheaper yet. In this country, with few exceptions, this can be interpreted as a case for concrete lining vs. other types. With the advent of new developments, this may not be true. Better concrete linings would do much to perpetuate their present favorable position.

Site conditions influence the choice of a lining. Hard-surface linings, such as concrete and shotcrete, are not suited to areas where soft foundation conditions cannot be corrected or areas where expansive soils are encountered. Since unlined canals usually have greater cross sections than would be required for canals with hard-surface linings, the use of membrane linings is frequently the solution to seepage control in established unlined canals. In some areas the cost of draining the area and preparing a satisfactory subgrade for a hard-surface lining within the boundaries of a canal where seepage is affecting adjacent land would be prohibitive. Membrane linings, on the other hand, particularly plastic film and butyl linings, can be installed satisfactorily on soft subgrades and in canals with some water standing on the bottom.

I am fully aware of the impact of the cost of irrigation improvements, having served as president of a canal company. Nevertheless, I should like to echo the words of a speaker at a meeting of the U.S. Bureau of Reclamation's low-cost canal lining committee in Denver.

2 years ago. In effect, he said, "I know you are charged with the responsibility of developing low-cost linings for canals, but I predict in the not-too-distant future we will be more concerned with better linings than lower cost linings." I feel sure this is coming. It will not be so much a matter of affording it but of affording anything less.

The trend will be toward linings that will give many years of service with a minimum of maintenance and closed conduits with built-in automation to regulate and distribute water. In my opinion, the use of buried pipe, surface gated pipe, and sprinkling systems will expand rapidly. This being the case, it will be good business in many areas to deliver water under pressure, particularly where topography lends itself to the development of hydrostatic pressure without the use of power. Deliveries might well be made through meters without restriction, controlling excessive use with a price structure for water to discourage overuse. To make such a system practical, storage along the distribution system will likely be required to meet the demand during periods of peak use. To make these storage structures practical, they must be watertight, which in many cases will mean lining.

What then does the future hold for us? What is our goal? What are low-cost linings? The canals of today are like the highways of yesterday. Greater efficiency of our conveyance systems cannot long be delayed. We who are charged with the responsibilities of these developments must project the requirements of the future, plan more adequately, and direct our research to the requirements of tomorrow.

DISCUSSION

Wednesday morning - First Session

Lawhon

I was very interested in Cy Lauritzen's philosophy in regard to our change in attitude of what the purpose of a canal lining is. Actually, I would say in the last 4 or 5 years, there has been more lining put in as a function for water control, rather than for water saving. In the area in which I work--which is to the east of here--and also a number of areas which, in planning stages, were not thought of as having seepage problems or developing seepage problems, as John Sutton pointed out yesterday, they are actually causing loss of land without any appreciable loss of water. An ideal example of that is the W. C. Austin project at Altus, Okla., where the head of the canal went through a sandy area--where in the planning stages the Bureau of Reclamation recognized they were going to have water loss and seepage problems so they put their canal lining test plot in this area--and yet the biggest problem right now on that project is in an area of heavy clays where the amount of water loss is exceptionally small; in fact, I do not think we could cut it down much with anything less than an impermeable membrane. And yet it is affecting some 5,000 acres, or about 10 percent of the project area by salting out. Now, of course, there is another little thing about this project. When it was designed, the canal would have a flow line which would have permitted the development of phreatic lines considerably below the land surface, but after turning it over to the Water Users' Association or the District, they decided

they needed about 2 feet more head on their canal; and in jacking up their canal to this extent, they raised their phreatic line and affected this 10 percent of the project area. I just hope that we won't lose sight of this fact, and I was glad to hear Cy's comments on this in regard to the functional use of linings for water control, as opposed to strictly for water savings. I am sure we don't have all the water-saving lining in yet, but I think it is time that we look at it from this other standpoint also. Thank you.

Kays:

I have a question for Mr. Willson. On this "buried" membrane chart you gave us, table 2 on page 12, it shows the polyethylene at $1.80 for 10 mil, and vinyl at $0.75 for 10 mil. I was wondering--that doesn't sound quite right.

Willson:

If you will notice, footnote 3 reads, "Cost represents construction by project forces and may not include equipment depreciation costs." These were experimental installations. The only reason we included them is to show you what the experimental linings would probably cost as a maximum. Actually, we were able to get the polyethylene for less cost than the polyvinyl, but the installation cost is about the same. As I remember, the last installation with polyvinyl cost us something like $0.42 a square yard, delivered in New Mexico. Polyethylene, I believe, could have been obtained for something in the neighborhood of about half more, but from our previous experience with the polyethylene and the polyvinyl, the polyvinyl is more durable in cold weather and easier to handle in placement in cold weather. Possibly Dr. Lauritzen could comment some on that. He has done more with the plastic.

Lauritzen:

Vinyl is tougher and is easier to seam and patch than polyethylene, but it is somewhat more expensive.

Wilder:

I would like to comment on table 2 that Mr. Willson presented, which has a world of worthwhile cost information in it. Actually, during the period of 1956-60 which you used as a basis for most of these costs, there were at least two fairly good-sized projects constructed by contract which involved 4-inch and 4 1/2-inch-thick unreinforced concrete lining. The 4 1/2-inch-thick job I am thinking of was in the Columbia Basin Project in Washington. I believe the contract was awarded early in 1958. I am speaking of Esquatzel diversion channel, which is not actually an irrigation canal but is very similar to one and it is a very favorable job because it involved, I believe, about 10 to 12 miles of uniform-size section. I believe the cost on the same basis that Mr. Willson used in this table, was a little over $3.00 per square yard for lining and trimming. The 4-inch-thick job I refer to was not a Bureau of Reclamation job, but was a Metropolitan Water District canal near Hemet, Calif. This canal lining is 4 inches thick, and again is a rather favorable kind of job because the canal section is uniform for quite a few miles. The unit cost of that lining, including the concrete and the trimming costs, I believe, was about $2.60 per square yard. These costs were both under the price which Willson shows for 3 1/2-inch-thick unreinforced.

I suspect this 3 1/2-inch price--I don't know where it came from--might be somewhat out of line, and if you want to do a little research yourselves on this table, you might calculate the incremental costs for each 1/2 inch you add to the lining. You will find that the incremental cost of 1/2 inch added to the 2-inch lining is $0.30. The 1/2 inch added to the 2 1/2 inches to make 3 inches is $0.26, and the next 1/2 inch incremental cost on this table from 3 to 3 1/2 is $0.94 per square yard. I don't think this is a reasonable kind of cost increase. Probably there is something wrong with the project on which that 3 1/2-inch cost is based.

Willson:

I think you are probably right. Footnote 1 says, "Costs are for contract construction over the period 1956-60, except as noted." Possibly the contractor unbalanced his bid for 3 1/2-inch lining.

Turner:

I would like to ask Cy Lauritzen what his experience has been with that 36-inch flexible pipeline? Did people "shoot holes" in it?

Lauritzen:

We did not have that problem, but we did have another problem. The steep embankments you saw in the slides sloughed off the second year, burying the tubing in places. The property on which this test was located changed hands the following year, so we had only a short period for evaluation. The little work we did, however, indicates that large size lay-flat tubing has possibilities for conveying water, if you can protect it from vandalism. Vandalism is also a problem with exposed membrane linings, water bags, and like structures. We even have trouble with people shooting through metal flumes. What you could do about that, I don't know, but I'm in favor of it.

Shaw:

I would like to ask Mr. Hanson--I noticed all this work was done with essentially no velocity--whether he would expect to get any great differences with velocity, particularly rate differences in velocity analysis and seepage losses?

Hanson:

Well, we haven't done any checking on that at all. I suppose the Laboratory over here by Tempe has more information on that.

Bouwer:

We have studied the effect of velocity on seepage for coarse material and fine gravel, and we could not detect any appreciable direct effects of velocity on seepage.

SEEPAGE REDUCTION WITH COLORADO CLAYS[1]

R. D. Dirmeyer, Jr. and M. M. Skinner

INTRODUCTION

A new research program on clay sealing was started in 1960 by the Colorado State University Experiment Station with guide principles as listed below:

1. Clays, locally available in Colorado, will be evaluated for sealing purposes by laboratory test procedures to be developed.
2. The best clays encountered in the laboratory testing will be evaluated in field trial installations in Colorado canals where (a) the seepage losses are high--in excess of 50 percent where possible, and (b) the previous bed materials are coarse--gravelly to rocky where possible.

These investigations are now nearing completion. In excess of 300 potential sediment-sealing clays representing approximately 100 deposits or sampling locations have been laboratory tested. In addition, 131 field trial installations with clay materials have been investigated, including 73 canals or ditches, 55 ponds, and 3 natural streams.

The information outlined in this paper is restricted to summary statements developed from the reports above. It is felt that this will suffice for most readers; however, should more detailed information be desired, please do not hesitate in requesting it from the authors of this paper.

This symposium, in itself, emphasizes that the canal lining needs of many irrigation systems are well recognized. It is also well recognized that large-scale efforts of seepage control are, in many instances, blocked or scaled down by the all-important factor of cost. Thus it may be seen that the idea of lowering the costs by using flowing canal water for placing an impermeable clay (or other sealant) in the leaky zones is an extremely attractive one. Judging from the widely variable results of such work, it may also be seen that many troublesome problems can be encountered in using the sediment-sealing methods.

Many materials can be used in sealing irrigation canals; and many methods of application are available. The summary statements that follow, however, are restricted to general items that are believed to apply equally as well to all silting and clay-sealing approaches. It is hoped that these will be of value in designing and controlling of clay-sealing applications.

CHARACTERISTICS OF ACCEPTABLE SEALING CLAYS

Impermeable.--Whether the material is called silt, clay, or bentonite, it must have one feature above all others--it must be watertight or, in other words, capable of holding water when used as a sealing agent in canals. This may seem like an overobvious statement, but during the course of our research activities, we have encountered many jobs

[1] Contribution from Colorado State University Experiment Station, Fort Collins, Colo.

(especially the silting examples) where the material used for silting or clay sealing apparently was not tested to see if it was impermeable--at least no record of the testing can be found.

Testing methods.--The various clay testing methods used in our research work are described elsewhere. In general, for uses where the clay is to be washed into place in the canal bed and banks, two of the tests are more important than the others: (1) the filter permeability and (2) mixibility index. The mixibility index gives a measure of the ease of mixing which is especially important if the clay is to be washed into suspension from dams spaced in the canal. The filter permeability gives a measure of the water-tightness after the clay is in place in the leaky zones of the canal bed and banks.

Chemistry and mineralogy.--In general, those clays of most favorable characteristics for sealing purposes are (1) high in content of colloidal-sized particles, (2) montmorillonitic or bentonitic in nature, and (3) 10 percent or more sodium saturated. They usually exhibit some degree of swelling upon wetting with water. This also means that the silting or clay-sealing methods have a limited use in hard-water areas, since a sodium clay when mixed or dispersed into hard water (high in calcium and magnesium) will exchange almost immediately to a calcium clay. When deposited as a filter cake, the calcium clay will usually be much more permeable than its sodium counterpart. This action can be counteracted by placing the clay mechanically as a compacted layer rather than by water as a sedimented material. It is difficult for the permeable structure to develop after the sodium clay is compacted.

CANAL CONDITIONS

Preparatory work.--Since the silting and clay-sealing methods are as implied, sealing methods only, they have no other significant benefits, such as the erosion and weed control benefits exhibited by hard-surfaced canal linings. Thus, before going ahead with the clay-sealing work, the channel should be clean, eroding areas repaired and riprapped, and any other needed maintenance work completed.

Stable channel.--Another factor, as vitally important as the clay characteristics and the channel preparation, is that the canal channel must be stable. If it is not, the life of the sealing is cut short. Conditions that will shorten the service life include:

(1) Erosion or undercutting of canal banks.
(2) Movement of bed material (as dunes) along the bottom.
(3) Burrowing or rooting of animals, such as crayfish, earthworms, muskrats, pigs, raccoons, etc.
(4) Growing or rotting of plant roots.
(5) Loosening of canal soil by frost heave or drying cracks.
(6) Careless canal cleaning.

Generally, the channels in fine-grained materials, such as sandy to clayey soils, will be much more disturbed by the above conditions than those in coarse materials, such as coarse sand, gravel, cobble, or talus.

SEALING OF COARSE MATERIALS

Experience has demonstrated that the best results with silting and clay sealing commonly are produced in canals traversing (or riprapped with) rocky to gravelly materials. The stability of the channel in such materials is usually excellent. Furthermore, the seepage rate is usually high. Thus, the immediate benefits of clay sealing in the coarse materials usually are of a much higher magnitude and they last longer than those for treatments in fine-grained soils.

Coarse material with fines.--Under ideal conditions, the coarse materials of the channel perimeter are underlain by a finer material in depth (say 6 to 12 inches deep). With this condition, the sealing takes place on the finer grained materials and is protected by the coarse-grained cover material. This ideal condition is not unusual. It is common in canals in mountainous areas where coarse materials are prevalent and where, with time, the flowing canal water removes the fines from the surface layer, leaving the coarse fragments as a gravel, cobble, or rock plating. The sealing produced under these conditions is protected from erosion and resists fairly well disturbing actions such as freezing and drying.

Coarse materials without fines.--If the coarse material is very open and lacks the necessary fines, the sealing clay may penetrate very well, but not seal. In this case, such as in canals traversing talus slopes, the intermediate particle sizes in the silt and sand range are needed as bridging agents and must, therefore, be furnished along with the sealing clay in order to produce an adequate sealing action. Wet sawdust has been used successfully as the bridging or void-plugging agent in remote alpine areas where sand and silt-size materials are not readily available.

Costs-to-benefits.--The most favorable ratio of costs-to-benefits from clay sealing was found in the mountainous areas in Colorado, where it is not uncommon to encounter canals that have from 50 to 100 percent loss late in the summer--at a time when the water is most needed. In these areas, conditions may be unfavorable for conventional canal linings because of higher-than-normal construction costs. In these same areas, however, conditions are commonly very favorable for clay sealing. Rocky to gravelly materials with high seepage losses are common in such areas, with the result that the benefits of clay sealing may be striking. For example, several installations have been noted where water was delivered where it could not be delivered before the clay treatment. The costs of some sedimenting installations have been recovered by benefits during the first season following the clay-sealing treatment.

SEALING OF FINE MATERIALS

In contrast to the generally good results produced in coarse materials, the least satisfactory results of clay sealing or silting have been produced in silty to sandy materials. This results from a combination of several factors:

1. Surface sealing.--The silting or clay-sealing materials will not readily penetrate many of the fine-grained soils. Thus, a surface coating of clay on the canal materials is a common result. This coating is vulnerable to puncturing by animals and plants, cracking by drying or freezing, or removal by water or wind erosion.
2. Unstable channel.--In addition to the problems of a surface coating, many of the canals in the fine-grained materials have unstable sections. Waves and flowing water cut the banks. Animals work the soil of the canal section. Sand dunes move along the canal bottom, and the soils crack down upon drying or freezing and thawing.

In addition to the above problems, it is difficult to measure accurately the seepage losses in canals with fine-grained soils. While seepage damage below canals in fine-grained soils may be very obvious, the amounts of water actually lost from the canals in these soils may be fairly low if compared to the losses from channels in coarse materials. In attempting to measure low rates of seepage loss, the limitations in accuracy of canal flow measurement are encountered. It is found that few water-measuring methods have an accuracy of

± 2.0 percent, and most methods probably are in the accuracy range of ± 5.0 to ± 10.0 percent. Since the seepage losses from fine-grained soils are often of the same magnitude as the error in seepage measurement for short lengths of canal (say less than 2 miles long), one is never quite certain whether the measured seepage loss is fact or fantasy.

Costs-to-benefits are, therefore, difficult to determine for clay-sealing work in fine-grained soils. It may be assumed, however, that in general it will take a special set of conditions to produce favorable results in fine-grained soils. For example, in an irrigation system where short supplies of water may require intermittent operation of the canals late in the summer, clay sealing with the first water into the dry canal can, under some circumstances, save sufficient water in one or two days of operation to pay for the clay used.

MAINTENANCE OF CLAY-SEALED CHANNELS

The maintenance requirements of canal linings are frequently overlooked, but the completion of such work on a periodic basis is, nevertheless, important. This is especially true for clay-sealing installations, since in several ways they are more susceptible to damage than conventional canal linings. The clay-sealed channels in rocky to gravelly materials generally will require less maintenance than those in the fine-grained soils, but in both cases repeat or followup treatments are recommended. The best time for the repeat treatments is in the spring with the first water into the dry canal. The amount of re-treatment will vary with the conditions, but the usual recommendation is for a 10 percent treatment--if 200 tons were used initially, 20 tons is recommended as the re-treatment each spring. In gravelly or rocky channels re-treatment may not be needed every year--perhaps every other year or every third year will be sufficient.

In some instances, special clay mixtures have been used as a maintenance procedure to seal leaky pipelines or canal linings.

RESEARCH NEEDS

During the course of our research work and also in reviewing the various examples of silting and clay sealing, a variety of research needs have become apparent--some directly related to the subject of canal sealing and some only indirectly related.

Of direct importance is the need for basic research into the various factors influencing canal seepage. The kind of canal bed material is a major factor, but there are a number of other important factors affecting seepage rates. The flowing canal water erodes bank or bed material in one area and deposits it it another. Animal life of many forms and activities inhabits and modifies the canal bed and bank materials. With changing condiditons of water, such as temperature, sediment content, hardness, etc., the seepage rate will change--but how much and why?

Clays are extremely varied in composition and in their permeability characteristics. Thus, there is a great need at the present time not only for fundamental studies of clays and their sealing characteristics but also for studies of the mechanics involved when all of the conditions of the canal site and clay sediment are interacting.

CONCRETE CANAL LININGS FOR CONTROL OF SEEPAGE[1]

Carl R. Wilder[2]

Previous speakers have described the consequences of seepage from irrigation canals. The loss of water is of prime importance, particularly in the water-deficient areas of Arizona and California. Also important is the economic loss when land adjacent to leaking canals becomes waterlogged and unsuitable for cultivation.

It has been found that lining irrigation canals reduces seepage loss. Such reductions in seepage often will more than pay for the lining.

Earlier speakers have told you about several types of irrigation canal linings and other methods of reducing seepage from canals. I would like to describe to you some recent developments in the construction of concrete canal linings, outline the desirable qualities of concrete for such linings, and review with you principles and methods which should be followed to attain those qualities.

Literally thousands of miles of irrigation canal of the type and size shown in figure 1 have been constructed in all areas of the west during the past 15 years. In some instances, placement rates of up to one mile per hour have been attained. Similar mechanized equipment, on a larger scale, has been used to place concrete linings in the larger supply canals (1, 10).[3]

It is a remarkable fact that through mechanized operations such as these the first cost of concrete canal lining has been held reasonably constant since 1945, despite a doubling of construction costs in general (10).

Figure 2 shows the trend of construction costs for concrete canal lining during the 15-year period from 1946 through 1960. The different lines on this chart represent various thicknesses of lining in different locations, each line representing one thickness and general size of canal in one specific area. The dotted line indicates the steady rise in the Engineering News-Record construction cost index, from about 320 at the start of 1946 to over 800 at the end of 1960. Note that during this period the unit cost of concrete canal lining remained reasonably steady and actually decreased in some areas.

In many localities underground pipelines are favored over surface canals for conveying irrigation water. Such pipelines have certain advantages over open-lined canals, including no evaporation, better control of irrigation water, and no right-of-way requirement. In recent years the use of rubber gasket rather than mortar joints has been favored, particularly for larger size pipe or high-pressure irrigation systems. Such pipelines are highly leak resistant.

Many irrigation systems, particularly in Arizona and California, are utilizing unreinforced cast-in-place concrete pipe. Several methods of constructing such pipe, most of

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[1] Contribution from Portland Cement Association.

[2] Regional conservation engineer, Portland Cement Association, Los Angeles, Calif.

[3] Underscored numbers in parentheses refer to References at the end of this paper.

Figure 1.--Constructing concrete-lined canal with subgrade-supported slip form.

them patented, have been used. Such construction has a higher first cost than slip form-lined canals of equal capacity, but usually is less expensive than lines constructed of precast reinforced concrete pipe.

Concrete to be used in canal linings or irrigation structures should have certain desirable qualities, including (1) durability; (2) strength; (3) watertightness; (4) resistance to erosion; (5) smoothness; and (6) economy.

1. The foremost requisite of concrete for any use is "durability." In some areas this means a high degree of resistance to freezing and thawing. In most irrigation areas it also must be resistant to chemical deterioration due to the presence of sulfates in the soil or water.

2. Concrete for canal linings must be strong enough to bridge over soft areas or voids which may develop in the subgrade, and concrete for structures must be strong enough to resist the forces of the foundation backfill and possible surcharge loads and hydraulic pressures.

3. Concrete used in canal linings or hydraulic structures must be watertight. If such concrete is adequately durable it usually will also be adequately watertight. Necessary construction and contraction joints should be sealed with a suitable joint filler to prevent leakage (9).

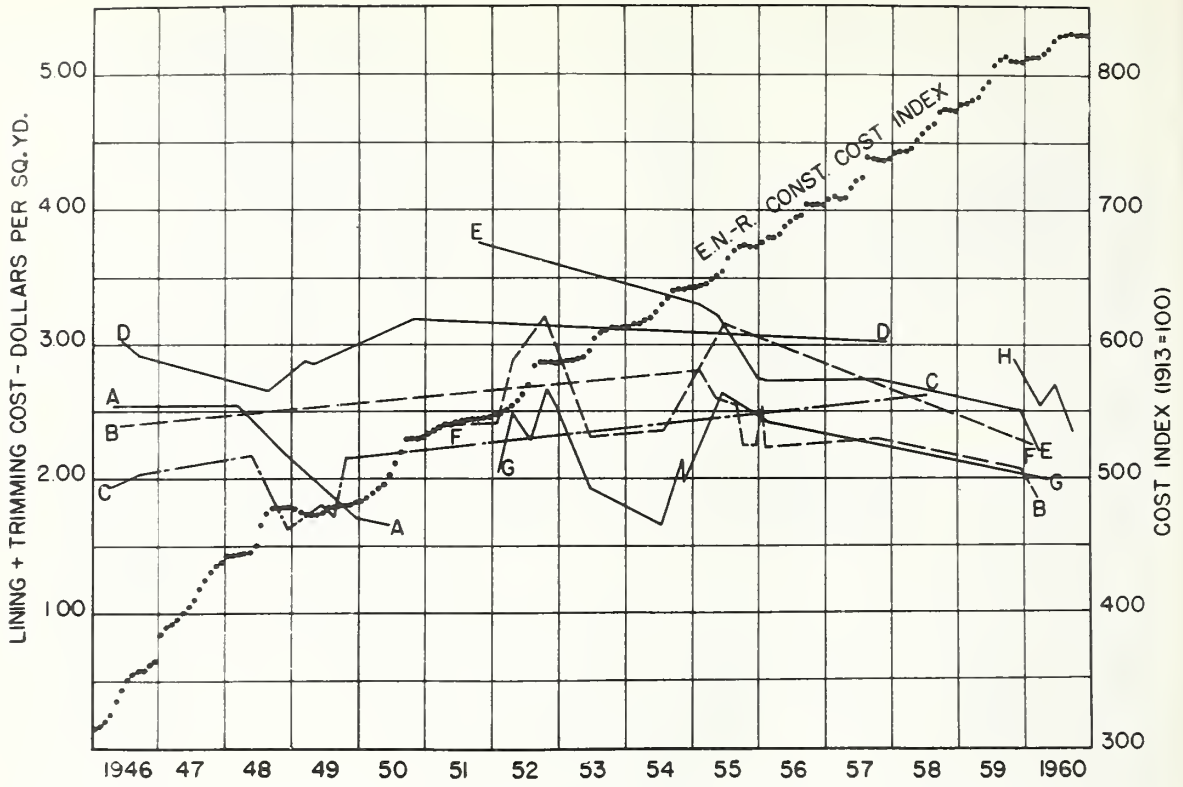

Figure 2.--Costs of concrete canal lining, 1946-60, are compared with Engineering News-Record construction cost index. Identification of lines: (A) Friant-Kern Canal, Calif., 3.5 in. thick; (B) Columbia Basin project, Wash., small canals, 2 in. thick; (C) Delta-Mendota Canal (USBR) and San Diego Aqueduct (MWD of SC), 4 in. thick; (D) Columbia Basin project, Wash., large canals, 4.5 in. thick; (E) Columbia Basin project, Wash., small canals, 2.5 in. thick; (F) Wellton-Mohawk Division, Ariz., small canals, 2.5 in. thick; (G) Wellton-Mohawk Division, Ariz., small canals, 2 in. thick; (H) Mercedes Irrigation District, Tex., small canals, 2.5 in. thick.

4. Resistance to erosion concerns us in those cases where high velocities or high sediment loads are encountered. Concrete that is of sufficient strength and durability will resist most erosive forces encountered in irrigation work.

5. For the low velocities ordinarily encountered in irrigation canals, extremely smooth surfaces are not necessary. Where velocities are high, or curvature considerable, care should be taken to prevent abrupt irregularities in the concrete surface. Usually such care will cost little.

6. Cost is not one of the basic physical qualities of concrete but should always be considered when designing a concrete mix for any use. We should always strive to attain the required durability and other desired qualities at the lowest possible overall cost.

Here are listed very briefly the three factors which are most necessary for producing quality concrete:

1. Suitable materials. It must be assumed that the aggregates used in making concrete are in themselves adequately strong and durable. Water must be reasonably clean and the correct type of cement should be used.

2. Adequate specifications, to correctly indicate the materials to be used, and how the work is to be done and to furnish a basis for inspection and acceptance of the work.

3. Quality workmanship.

Two commonly used methods for specifying concrete are described in American Society of Testing Materials Specifications C94, Ready-mixed Concrete. These may be summarized as follows:

Alternate 1	Alternate 2
Maximum water-cement ratio, gallons per sack	Minimum compressive strength at 28 days, pounds per square inch (laboratory-cured specimen)
Minimum cement content, sacks per cubic yard	

Both Alternates

Maximum size of coarse aggregate
Slump, minimum and maximum
Entrained air, minimum and maximum

Alternate 1 might be called a "method" specification, whereas alternate 2 is an "end result" or performance specification. The alternate 1 method is preferable, since there are so many factors other than strength to be considered in specifying concrete.

While concrete made with a given water-cement ratio can, by varying the aggregates, be made very stiff or very fluid, the slump of concrete for lining canals with sloping banks must be carefully controlled within close limits. Air-entrained concrete for use in canal-lining slip forms usually should have a slump of about 3 inches.

It is important to recognize the relation of strength to water-cement ratio. A mix with a water-cement ratio of 7.5 gallons per sack should produce a 28-day strength between 3,000 and 4,000 p.s.i., whereas for a water-cement ratio of 6 gallons per sack, the strength should be between 4,000 and 5,200 p.s.i.

In many irrigation areas concrete must be placed during periods of relatively high temperature. The water requirement per cubic yard of a concrete mix increases as the temperature of the concrete increases. If we are working with a mix having 1.5-in. maximum size coarse aggregate, the water required to maintain a 3-in. slump would be about 32.5 gal. per cu. yd. at 50°F. but 35.5 gal. at 90°F. This indicates the desirability of preventing undue temperature rise in the concrete as it is being mixed and placed. It is obvious that if the water content of a mix has to be increased, on account of temperature or for any other reason, to maintain the desired workability the cement content must be increased correspondingly.

In irrigation canals, shrinkage which can open up cracks in the lining is very undesirable. All concrete shrinks as it hardens and cures but steps can be taken to minimize this shrinkage and its consequences. To prevent random shrinkage cracks in concrete canal linings, contraction control joints should be placed at intervals of 10 to 15 feet, the shorter intervals being used with thinner linings. Contraction joints usually are of the weakened-plane type formed by constructing a vertical groove in the top third of the lining. To prevent leakage of water through the joint, it should be filled with a suitable sealing compound before the canal is placed in operation (9).

Cracking that occurs in the surface of fresh concrete soon after it is placed and while it is still plastic is called "plastic cracking." In irrigation canals plastic cracking usually can be prevented by (6):

1. Moistening the subgrade just ahead of the lining operation.
2. Lowering the temperature of the concrete in hot weather.
3. Applying a curing membrane immediately after completion of finishing operations.

One of the most important parts of an adequate concrete specification is the paragraph on curing. Here are listed some of the more important effects of adequate curing on the properties of hardened concrete:

1. Durability is greatly improved.
2. Strength gain is accelerated.
3. Early cracking is eliminated.
4. Wear resistance is greatly improved.
5. Watertightness is greatly improved.

Figure 3 shows the effect of curing on the strength of concrete. Note that concrete which is moist cured for 14 days will have a 28-day strength, about twice that of concrete which is allowed to dry out in the air. Omitting curing is just about as sensible and has about the same effect as leaving out from one-third to one-half of the cement.

To provide adequate curing, the concrete must be kept saturated continuously. This may be accomplished by sealing in the water, which is a part of the mix, by means of a sprayed-on curing compound or other impervious membrane, or by keeping the exposed

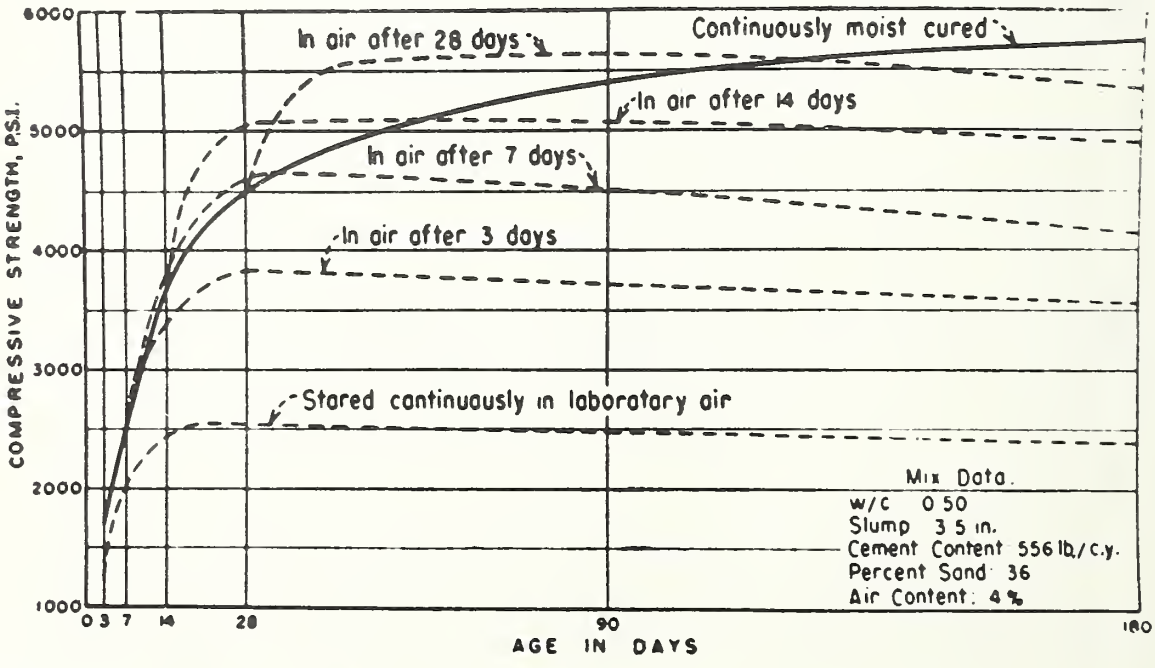

Figure 3 —Compressive strength of concrete dried in laboratory air after preliminary moist curing.

surface of the concrete continuously moist during the curing period, usually 3 to 5 days. Adequate strength gain and development of durable and wear-resistant surfaces will be accomplished if the air temperature above the lining is at least 70°F. for the first 3 days or 50°F. for the first 5 days of the curing period.

In areas where winter temperatures drop below freezing, the resistance of concrete to freezing and thawing is of primary importance. The durability of concrete is greatly improved by the entrainment of air in the concrete. In most parts of the country air-entrained concrete is well known and extensively used. I wish I could emphasize in such a way that it will never be forgotten that good concrete--the kind of concrete you all want to make--should include air entrainment. Since 1945 all Bureau of Reclamation concrete has been air entrained. The Corps of Engineers, practically all state highway departments, and more and more private engineers are specifying air entrainment. Air-entrained concrete is as easy to make as non-air-entrained concrete. The additional cost is little or nothing; the benefits are tremendous.

In addition to improving the resistance of concrete to freezing and thawing, air entrainment also improves the sulfate resistance of hardened concrete. The bubbles of entrained air also have significant beneficial effects on plastic concrete. Air-entrained concrete is cohesive and looks and feels "fatty." Segregation and bleeding are markedly reduced. Workability is improved, even in lean mixes or mixes with poorly shaped aggregates that otherwise would be harsh and difficult to work. Because of this improved workability, water and sand contents of air-entrained concrete can be significantly reduced.

Much of the information I have presented to you on cement content and air entrainment is based on laboratory tests. Some years ago it was suggested that these laboratory indications should be verified by field tests. Accordingly, as part of the Long-Time Study (L.S.T.) of Cement Performance in Concrete several field installations were made. Figure 4 shows the L.T.S. concrete "farm" near Naperville, Ill. The concrete specimens were made with various cements, aggregates, and water-cement ratios. Several of the specimens will be discussed in detail.

Figure 5 shows three of the concrete boxes in that farm, which were built in October 1941 and photographed in September 1959. These were made with identical cement type and brand, the same aggregates, and equal slumps. The only variable for boxes 11 D6 and 11 D3 was the cement content, which determined the water-cement ratio. The cement content was 4.5 sacks per cu. yd. for box 11 D6 and 6 sacks per cu. yd. for box 11 D3. This illustrates the effect of the water-cement ratio on the durability of concrete, for if all other conditions remain equal, higher cement contents mean lower water-cement ratios.

The foregoing example might leave you with the impression that all that is necessary to make good concrete is to use less water per bag of cement. However, water-cement ratio by itself does not tell the whole story.

The third box shown in figure 5, box 11 TD6, was made with the same cement and aggregates, the same proportions, and the same slump as 11 D6, but with an air-entraining agent added to the mix. The superiority of this concrete from the durability standpoint certainly is obvious. Today, 22 years after they were built, the 84 boxes in this farm which were made with air-entrained concrete were practically perfect.

The long-time study of cement performance in concrete also includes a test plot near Sacramento, Calif., where concrete made with various cements, aggregates, and water-cement ratios are exposed to sulfate soils and water. These sulfates usually make

Figure 4.--Two views of test farm at Naperville, Ill., a part of the long-time study of cement performance in concrete.

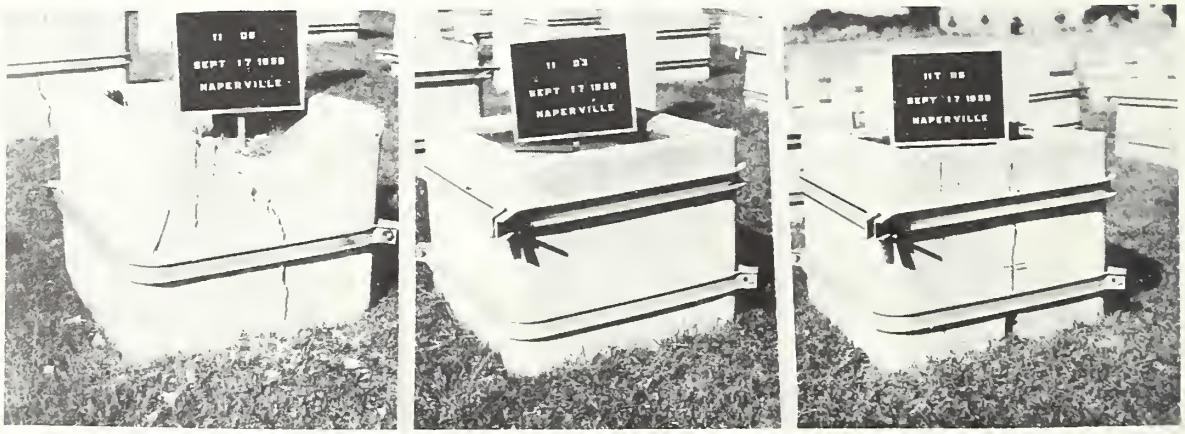

Figure 5.--Three of the test boxes at L.T.S. Naperville farm. Constructed October 1941, photographed September 1959. Concrete in box 11 D6 has 4.5 sacks of cement per cu. yd., 8-in. slump. Box 11 D3 has 6 sacks per cu. yd., same slump, hence higher water-cement ratio. All other materials and quantities identical. Concrete in box 11T D6 has same quantities and proportions as 11D6, except that concrete contained entrained air.

themselves apparent by the "white-alkali" deposits seen near many canals, drains, and ponds. Results to date show that rich mix concretes with lower water-cement ratios are more resistant to sulfate attack than lean mixes or higher water-cement ratio concrete; Type II and Type V cements are more resistant than Type I or Type III; and air entrainment likewise helps increase sulfate resistance. Type II cement is intended for use where sulfate concentrations are higher than normal but not unusually severe. Type V cement should be used where concrete is exposed to severe sulfate attack (3).

Now how can we obtain these qualities economically? We first must determine a suitable water-cement ratio (3) and then select mix proportions that will deliver concrete of the desired quality. In general, economy will be served by using the largest maximum size aggregate that can be accommodated in the concrete to be placed. For canal linings placed by slip form the maximum aggregate size probably should not be more than one-half the thickness of the lining.

Figure 6 shows how the cement requirement of a concrete mix varies with the maximum size of the aggregate used. For example, it may be your custom to use 3/4-in. aggregate, in which case this mix, with a 3- to 5-in. slump and 6.5-gal. per sack water-cement ratio would require about 6 sacks of cement per cu. yd. of concrete. However, if conditions were such that a larger size aggregate, perhaps 3-in. maximum, could be used, the cement requirement for equal durability, strength, and workability would be only about 5 sacks per cu. yd. If very many cubic yards of concrete are required, such a saving would be very important.

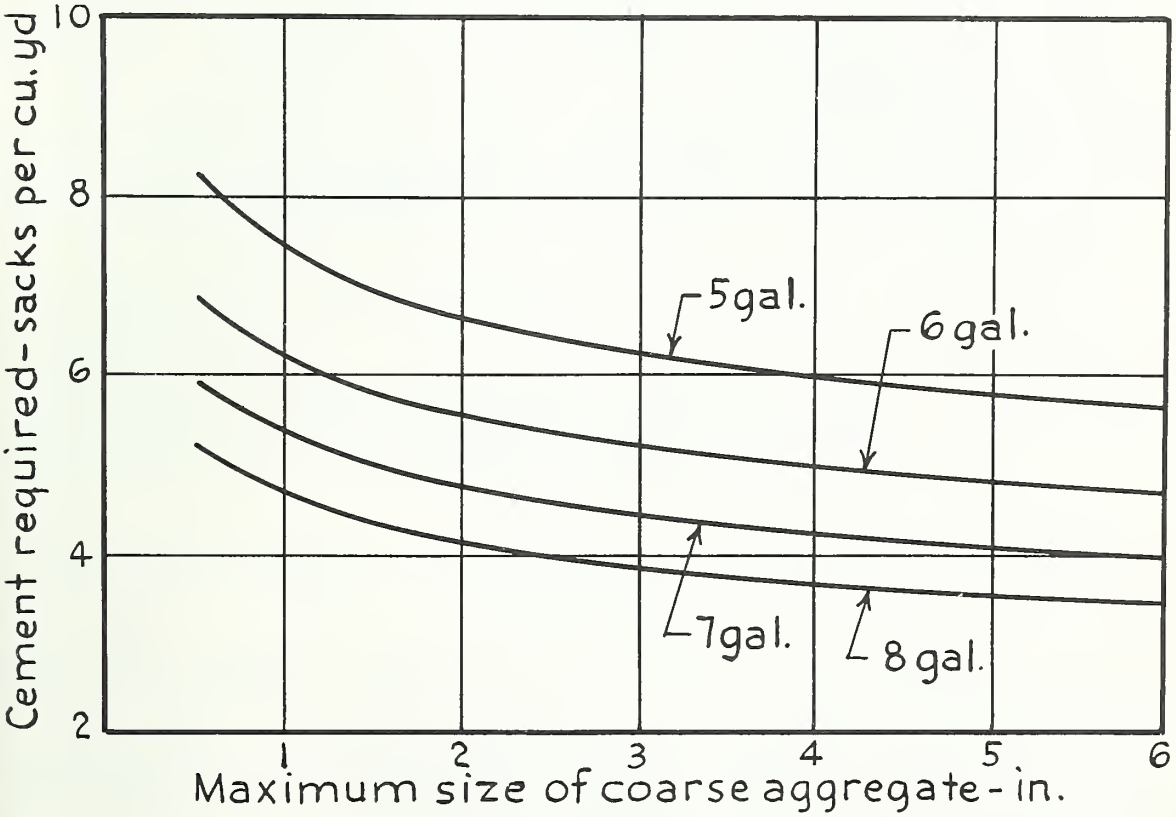

Figure 6.--Cement requirement of a concrete mix varies with the maximum aggregate size for various water-cement ratios.

It is hoped that you will not infer from this discussion of water-cement ratios, air entrainment, and sulfate attack that making good concrete is a tremendously difficult and hazardous undertaking. Actually, the rules are few and simple and relatively easy to follow. The Bureau of Reclamation Concrete Manual and Portland Cement Association publications on quality concrete (which are available on request in the United States and Canada) set forth these rules clearly and concisely. We hope that you will find them useful and that they will help you make lots of good concrete.

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SEALING SMALL RESERVOIRS WITH CHEMICAL SOIL DISPERSANTS[1]

Rey S. Decker[2]

INTRODUCTION

Testing and evaluating soil permeability characteristics and providing recommendations for treatment to reduce seepage losses from small irrigation and stock water reservoirs are important phases of Soil Conservation Service technical assistance to Soil Conservation District cooperators. Natural soil materials involved in reservoir sealing problems cover the entire range of soil composition from coarse sands and gravels to highly plastic silts and clays. Treatment recommendations and applications generally

[1] Contribution from the Soil Conservation Service, U.S. Department of Agriculture.

[2] Head, Soil Mechanics Laboratory, Lincoln, Nebr.

involve the use of commercial and native or locally occurring bentonitic clay or compacted earth blankets for coarse-grained materials and the use of common salt (soldium chloride) or sodium polyphosphate salts for fine-grained soils.

This paper deals only with the use of salt and sodium polyphosphates as sealing agents.

Our records show that the use of salt to reduce seepage from stock ponds was recommended by Soil Conservation technicians in 1939 and that several ponds in New Mexico and Arizona were treated in 1940 with varying degrees of success.

Since 1943, laboratory tests for sealing recommendations have been conducted on samples from over 1,000 small reservoirs. More than half of these projects involved the use of salt or sodium polyphosphate with the polyphosphates dominating the recommendations after 1954.

The number of ponds that were actually treated in accordance with laboratory recommendations is not known, nor is it known how many ponds were treated without laboratory tests. It is known, however, that most of the pond sealing work done in the past 5 years has been based upon local evaluation of soil characteristics and actual performance of similar materials in the immediate area.

Unfortunately, many of the early laboratory testing records are not available or not complete with respect to present standards of testing.

Very few data are available showing actual measured seepage losses either before or after treatment. Sufficient testimonials are available, however, as shown in General Recommendations, which precede Literature Cited, to indicate that most of the sealing work done with Soil Conservation Service technical assistance has produced results which are satisfactory to the individual concerned.

FACTORS AFFECTING PERMEABILITY

There are many factors which affect the permeability of soil materials. Gradation characteristics with respect to size and distribution of mineral particles and the void ratio or density of the soil mass affect the size and number of pores available for water conduction in all classes of soil, coarse-grained or fine-grained.

There is one important factor peculiar to fine-grained soils that is highly significant with respect to permeability: the characteristics and stability of aggregates or granules, or the "structure" of plastic soils as it pertains to the arrangement of mineral particles held together by chemical or electrochemical forces to form grains or granules much larger than the individual particles.

It is an established fact that a well-aggregated clay soil has much higher permeability than the same soil in a nonaggregated state.

The nature and arrangement of soil aggregates are greatly influenced by the characteristics of the clay minerals present in the soil. The fact that the properties of the clay minerals can be greatly altered by the action of small amounts of certain chemicals provides the basis for successful use of soil dispersants to reduce permeability.

Studies on the use of chemicals and organic matter to increase or stabilize the aggregating properties of plastic soils to improve the agronomic features date back to the early 1900's. The beneficial effects of reducing the size and changing the character of aggregates in soils used for certain engineering structures have also been recognized for many years. Reports of puddling and tamping clay soils to reduce the permeability of earth dams are historical.

In recent years, much research on engineering properties of fine-grained plastic soils has centered on the formation, structure, and stability of the soil aggregates.

Rosenquist (12)[3] summarizes the theories and research presented by Terzaghi, Goldschmidt, Lambe, and others on the mineral arrangements of clay soils. Although these investigators do not agree in all aspects, it can be concluded that the clay minerals in most aggregated clay soils are arranged at random with end-to-plate or end-to-end contacts to form an open, porous card house or honeycomb structure. Application of compressive or electrochemical forces to these porous aggregates may result in collapse or rearrangement of the open structure and reorientation of clay particles with plate-to-plate contact and a certain degree of parallelism. Lambe (6) refers to this parallel orientation as dispersed structure.

The theories regarding development and stability of clay aggregates and other clay properties such as plasticity and cohesion involve details of clay mineralogy beyond the scope of this paper. The subject is well covered in a comprehensive text by Grim (4). Some of the aspects of clay mineralogy pertinent to the subject of chemical dispersants may be summarized as follows:

1. Clay minerals are crystals primarily composed of plate-like molecules of alumina and silica. The arrangement and size of these plates vary with kind of clay mineral. Montmorillonite crystals are very small with approximate dimensions of 14 to 20A thick by 200 to 2000 A. The molecular plates are so arranged that the crystal lattice is expandable. Kaolinite is a larger crystal with thickness of 500 to 2000 A by 3000 to 40,000 A. The arrangement of the particles is such that the crystal lattice is nonexpandable. The relative size of the different clay minerals is important since specific surface of the particles affects other clay properties. One gram of montmorillonite would have about 8500 sq. ft. of specific surface, whereas 1 gram of kaolinite would have about 100 sq. ft. of specific surface.

2. The clay minerals carry net electrical charges (principally negative but some positive) and establish an electrical field around each particle or micelle. The negative charge carried by the particle varies with the type of clay mineral. Lambe (8) gives approximate values of montmorillonite carrying some 15,000 ions per particle and kaolinite some 4,000,000 ions per particle.

3. The electrical charges carried by the clay minerals are balanced by ions at the surface of the crystal which are replaceable or exchangeable for other ions in an aqueous solution. The cation exchange capacity of clay minerals varies from 80 to 150 meq./100 g. for montmorillonite to 3 to 15 meq./100 g. for kaolinite with the other common mineral illite showing 10 to 40 meq./100 g. (Grim, 4) Calcium is the dominant exchangeable cation in many soils. This is certainly not true in many of our western arid (alkali) soils where sodium occupies a prominent part of the exchange complex.

[3] Underscored numbers in parentheses refer to Literature Cited at the end of this paper.

The replaceability of exchangeable cations varies with many factors: concentration of replacing ions, nature of anions in replacing solution, nature of the clay mineral and others. According to Grim (4), the higher the valence of the ion, the greater its replacing power and the harder it is to displace, all other things being equal. Thus,

$$Li > Na > H > Ca > Mg.$$

4. Clay minerals have the power to adsorb water, to balance the net charge on the surface and to hydrate the adsorbed cations. The thickness of the layer of adsorbed water depends upon the kind of clay and the hydration capacity of the adsorbed ions. This adsorption of water and hydration of exchangeable cations results in great increase in the effective or active size of the clay micelle. The radius of a hydrated sodium cation is about seven times that of a dry cation. The thickness of the oriented water layer around the clay particle has a marked effect on plasticity and cohesion of soil. Grim (4) states that with hydrated sodium montmorillonite, some orientation of water may be in effect as far as 100 A out from the micelle, but with calcium montmorillonite, the orientation of water does not extend beyond 15 A from the micelle. This property of hydrated sodium cations undoubtedly accounts for the fact that sodium clays require less water to develop plasticity than do calcium clays.

   In montmorillonite clays (expanding crystal lattice), water is also adsorbed within the lattice, thus producing significant swelling of the clay crystal. This swelling phenomenon does not occur with the two-layer clays such as kaolinite.

Rosenquist (11), Michaels (10), and others have reported on the properties of clay-water systems in evaluating the factors that influence formation and characteristics of structure of clay. These discussions lead to the following conclusions regarding structure formation:

Dispersed structure of clays is promoted by--

1. Decrease in electrolyte concentration.
2. Decrease in valence of adsorbed cations (Li or Na for Ca and Mg).
3. Increase in size of hydrated ion (Na over Ca).
4. Increase in pH of solute.
5. Increase in anion adsorption.

The relative effects of clay structure on some of the engineering properties of soils may be listed as follows:

Engineering properties:	Aggregated or card house structure	Dispersed structure
Unit dry density	Lower	Higher
Consolidation potential	High	Low
Permeability	Higher	Lower

It is apparent from the foregoing discussion that the addition of small amounts of monovalent soluble salts to soils containing calcium-saturated clay minerals should reduce the amount of aggregation with end result of lower permeability.

THE EFFECTIVENESS OF SODIUM CHLORIDE AND SODIUM POLYPHOSPHATE AS DISPERSANTS AND SEALING AGENTS

There are many soluble salts which meet the requirements of a dispersing agent. The sodium salts are most commonly considered. Those sodium salts with polyphosphate anions are exceptionally effective.

Effect on Liquid Limit

Lambe (7) rates the effectiveness of dispersants on the amount of reduction in the liquid limit (L.L.). The degree of reduction varies with the character of the clay minerals as well as the type of dispersant. With additions of 0.10 percent of dry soil weight of tetrasodium polyphosphate (TSPP), he shows reduction in liquid limit from 50 to 45 for Boston Blue clay and from 46 to 45 for kaolinitic Virginia sandy clay.

Test results in our laboratory show the same tendencies shown in table 1.

Table 1.--Effect of tetrasodium polyphosphate (TSPP) on liquid limit (L.L.)

TSPP application			Liquid limit for--		
Lb. sq. ft.		Percent by dry weight of soil	Virginia limestone soil < 0.002 = 45 percent < 0.074 = 90 percent	Kentucky Maury series < 0.002 = 51 percent < 0.074 = 80 percent P.I.[1] 22	Indiana glacial drift soil, Russell series < 0.002 = 16 percent < 0.074 = 90 percent P.I.[1] 9.4
			Percent	Percent	Percent
0.00	=	0.	45	56	33
0.05	≅	.10	40	54	29
0.10	≅	.20	40	49	--

[1] Plasticity index.

The effect of dispersants on the liquid limit provides a valuable field test to evaluate probable effectiveness of sealing with polyphosphate salts (14). The salt is an effective dispersant if a small handful of moist soil becomes noticeably wetter after adding a pinch of the dispersing agent and thoroughly kneading the mixture.

Gradation and Amount of Fines

Data in figure 1 on gradation characteristics of soils which have been effectively sealed in accordance with Soil Conservation Service recommendations cover a range of 12 to 58 percent material finer than 0.002-mm. diameter and 50 to 100 percent finer than 0.074-mm. diameter.

It is concluded that soils should contain at least 15 percent material finer than 0.002 mm. and 50 percent finer than 0.074 mm. to produce an effective reduction in seepage losses when treated with sodium polyphosphate or sodium chloride. Other methods of treatment would be better for coarser soils.

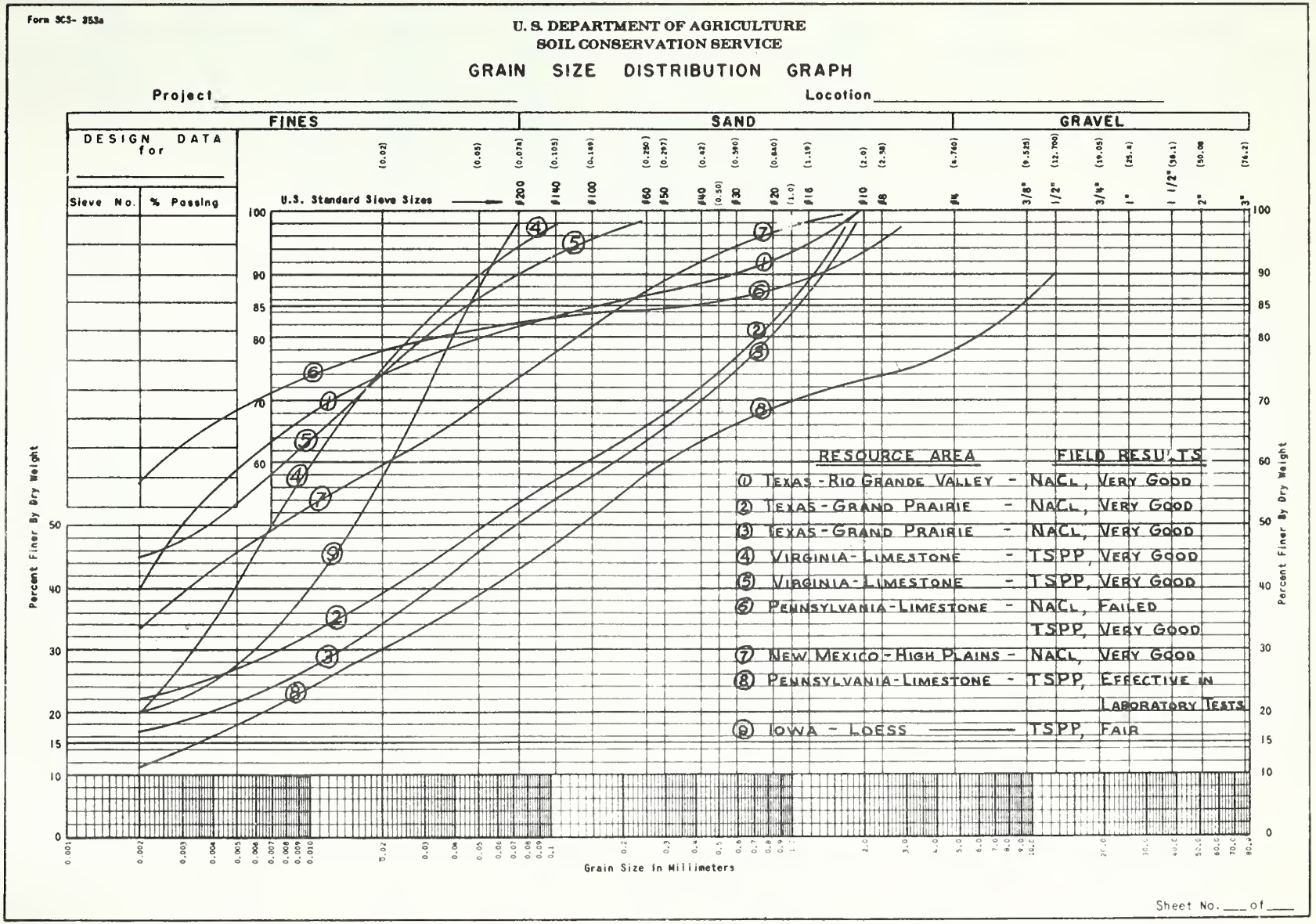

Figure 1.--Range in gradation of soils from ponds effectively sealed.

Nature of Clay Minerals and Ion Exchange Complex

The importance of the type of the clay mineral and the reactions in the exchange complex was forcibly emphasized in Soil Conservation Service operations a few years ago. Several ponds in the Northeastern States were treated with salt without success, although satisfactory results were obtained in the laboratory. Some of the failures were attributable to poor mechanics of treatment but part of the trouble undoubtedly was due to the difference in cation exchange reactions between salt and montmorillonite vs. kaolinite.

As pointed out previously most of our early sealing work was done in the area with less than 25 inches of rainfall. According to Alexander, Hendricks, and Nelson (1), the fine-grained soils in this area generally contain significant amounts of montmorillonite clay whereas the clay minerals in soils of many Northeastern States are dominantly kaolin.

The effect of the exchange of Ca for Na without regard to anionic reactions brought about by the addition of NaCl to montmorillonite clay along with swelling of the particles apparently produces sufficient dispersion for satisfactory sealing. One of the end products of this exchange is $CaCl_2$, which may stay in the clay-water system in ionized form as a continuing source of calcium. This evidently is not detrimental as long as there is an adequate supply of NaCl in the system.

In the case of the kaolin clays of the Northeast, we are dealing with nonswelling materials with relatively little exchange capacity and much less adsorbed or oriented water. Satisfactory sealing of these materials must depend upon obtaining as much dispersion as possible and removing the reaction products from the ionic solution.

Kaolin clay has a high capacity for adsorbing phosphate ions. This feature, coupled with the formation of nonionized or insoluble calcium polyphosphates as an end product of the Ca - Na exchange reaction with sodium polyphosphates, evidently produces sufficient dispersion and clogging of pore space to effect significant reduction in permeability.

Some investigators report that in the dispersion of calcium kaolinite, only a small part of the exchangeable calcium on the clay is replaced by sodium and that this exchange alone does not cause dispersion. Trexler[4] reports that work in their laboratory shows polyphosphates effective in dispersing minerals which have no ion exchange capacity.

Data in table 2 showing cation exchange relationships before and after treatment with TSPP and salt certainly follow the above discussion. Exchange capacities of the soils tested from New York, Pennsylvania, and Virginia range from 7 to 16 meq./100 g. soil. Exchangeable sodium from TSPP ranged from 9 to 36 percent of the total exchange capacity. The sodium percentage exchange from NaCl was considerably higher (25 to 42 percent) than the replacement from TSPP for all but one sample, yet NaCl did not produce satisfactory seals on some of these and similar soils.

Many investigators report that 20 to 30 percent sodium in the exchange complex produces dispersed soils with low permeabilities. Our data indicate this is not necessarily true for kaolinitic soils.

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[4] Trexler, M. V. Private communication. Westvaco Mineral Products Division, Food Machinery & Chemical Corp., New York. 1956.

Table 2.--Cation exchange relationships before and after treatment with sodium dispersants

Location and Soil class	Percent finer than 0.002 mm.	Before chemical treatment[2]						After chemical treatment					
		L.L.	P.I.	Exchangeable cations				Exch. Na from TSPP		Exch. Na from NaCl		Permeability	
				Total	Ca	Mg	Na					TSPP	NaCl
				Meq./100 g. soil				Meq./ 100 g. soil	Percent of total	Meq./ 100 g. soil	Percent of total	Ft./ day	Ft./ day
N.Y.--ML	18	--	--	10.74	8.83	0.38	0.52	1.68	16	2.68	25	0.001	0.0002
N.Y.--CL	21	--	--	15.65	11.20	1.24	.55	2.11	13	4.30	27	.001	.002
N.Y.--CL	21	--	--	15.50	9.23	.56	.56	1.80	11	4.40	28	.0002	.001
Penn.-ML	24	--	--	10.80	1.50	1.06	.45	2.30	21	3.00	28	.00001	.0003
Penn.-ML	24	34	9	7.37	3.79	1.16	.56	2.70	36	3.15	42	.0003	.00001
Penn.-MH	47	44	15	16.00	3.46	.92	.67	1.43	9	1.59	10	.001	.04
Va.---ML	20	32	7	11.60	4.89	.46	.45					.0003	
Va.---CL	33	34	12	11.60	4.98	.80	.48					.00001	
Va.---CL	38	39	15	13.0	5.81	.39	.49					.00001	

[1] Soil class symbols are those given in The Unified Soil Classification System, Tech. Memo. 3-357, Waterways Experiment Station, v. 1. 1953.
[2] L.L = liquid limit; P.I. = plasticity index.
[3] TSPP = Tetrasodium polyphosphate.

Although most of our pond sealing work today involves the use of polyphosphates, we are still interested in the use of salt where it is feasible.

Skempton's (13) work on relationship of plasticity index and amount of clay or "activity" (P.I./percent < 0.002) was reviewed as a possible means of eavaluating the dominant type of clay without complex analyses.

Skempton gives activity values for clay minerals of kaolinite, 0.33 to 0.46; illite, 0.9; Ca montmorillonite, 1.5; and Na montmorillonite, 7.2. He rates activity as:

Inactive clays - activity < 0.75
Normal clays - activity 0.75 - 1.25
Active clays - activity > 1.25

A plot of 0.002 for several soils, many of which were successfully sealed in the field, is shown in figure 2.

These data indicate that soils with activity > 0.6 and at least 15 percent finer than 0.002 mm. can generally be satisfactorily sealed with salt.

Effectiveness of Different Polyphosphates

There are a number of sodium polyphosphate products available as dispersants (table 3).

Limited laboratory tests were made on the effectiveness of these three compounds as sealing agents. These data are shown in Table 4. It appears that tetrasodium pyrophosphate (TSPP) is slightly better than the others for the soils tested but there is little difference between TSPP and sodium tripolyphosphate (STPP). The hexametaphosphate (SPP) was consistently less effective than the others.

According to these data, either TSPP or STPP is satisfactory with a slight preference for TSPP.

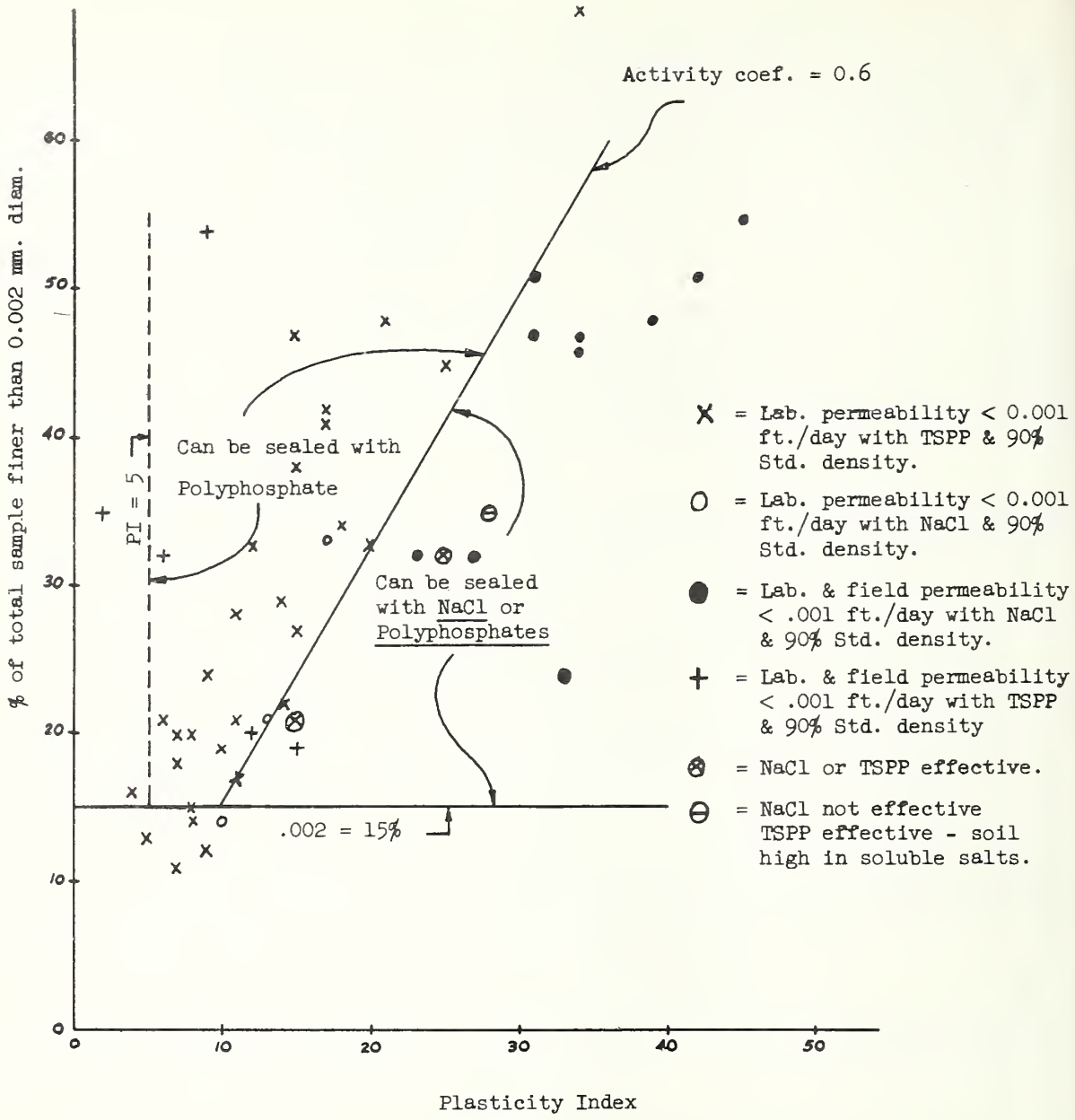
Activity coef. = 0.6
% of total sample finer than 0.002 mm. diam.
Plasticity Index
PI = 5
Can be sealed with Polyphosphate
Can be sealed with NaCl or Polyphosphates
.002 = 15%
= Lab. permeability < 0.001 ft./day with TSPP & 90% Std. density.
= Lab. permeability < 0.001 ft./day with NaCl & 90% Std. density.
= Lab. & field permeability < .001 ft./day with NaCl & 90% Std. density.
= Lab. & field permeability < .001 ft./day with TSPP & 90% Std. density
= NaCl or TSPP effective.
= NaCl not effective TSPP effective - soil high in soluble salts.

Figure 2.--Relationship of plasticity index and material finer than 0.002-mm. diameter for soils with dry tamp (85 percent standard) permeability > 0.01 ft./day and treated permeability < 0.001 ft./day.

Table 3.--Formula, solubility, and sodium content of three common dispersants

Dispersant	Formula	Na content	Solubility
		Percent	Percent at 75°F.
Tetrasodium pyrophosphate (TSPP).....	NaP_2O_7	47	7.5
Sodium polyphosphate or hexametaphosphate (SPP).....	$Na_{12}P_{10}O_{31}$	36	150
Sodium tripolyphosphate (STPP).....	$Na_5P_3O_{10}$	43	45

Table 4.--Comparison of the effects of three polyphosphate salts on permeability of compacted soil

Soil class[1]	Percent finer than 0.002 mm.	Permeability coefficient at 90-percent standard density	Reduction in permeability coefficient by polyphosphate additives with compaction to 90-percent maximum standard		
			TSPP[2]	SPP[2]	STPP[2]
		Ft./Day	Percent	Percent	Percent
CL	30	0.001	250	250	330
CL or ML	26	0.004	440	150	200
SM -- SC	20	0.004	660	360	570
SM	16	0.015	1600	1600	1600

[1] Symbols are those given in The Unified Soil Classification System, Tech. Memo. 3-357, Waterways Experiment Station, v. 1. 1953.
[2] TSPP = Tetrasodium pyrophosphate;
SPP = Sodium polyphosphate or sodium hexametaphosphate;
STPP = Sodium tripolyphosphate.

Rate of Application

Sodium Polyphosphate

Most of our work calls for applications of polyphosphates at a rate of 0.05 lb./sq. ft. This is equivalent to the 0.10 percent of dry soil weight recommended by Lambe and Anderson (9) for a 6-inch treated layer compacted to 100 pounds per cubic foot (p.c.f.) dry density.

Data in table 2 show exchange capacities of 7 to 16 meq./100 g. for some typical Northeastern soils.

TSPP or STPP applied at the rate of 0.05 lb./sq. ft. will provide about 2 meq. of sodium per 100 g. of dry soil which should be adequate to produce about 20 percent exchangeable Na on the clay complex of these kaolin-illite soils.

Laboratory tests show that applications of more than 0.05 lb./sq. ft. are necessary to seal some of the high exchange capacity (montmorillonite) soils.

Many of the Texas Blackland soils on which we have worked show exchange capacities of 40 or more meq./100 g. Applications of 0.1 lb./sq. ft. have given excellent laboratory results for these soils. This amount of TSPP would provide enough sodium theoretically to produce 10 percent or less exchangeable Na on soils with exchange capacities of 40 to 50 meq./100 g.

These results re-emphasize the importance of the clay-phosphate reactions as well as the clay-Na action in the highly effective dispersive action of sodium polyphosphates.

Sodium Chloride

With reference to figure 2, the use of salt should be restricted to the more active (higher exchange capacity) clays. Usual recommendations for salt applications range from 0.20 to 0.33 lb./sq. ft. These applications would provide enough sodium to produce about 20 percent sodium on the exchange complex of soils having exchange capacities of 35 to 55 meq./100 g.

It should be emphasized that laboratory testing is required to determine the best kind and rate of soil dispersant to be used on a particular soil unless sufficient data are available on similarity of composition and field performance of previously treated soils.

Thickness and Compaction of Treated Blanket

Field and laboratory experiences show that thoroughly mixing the chemical dispersant with the soil is necessary for effective sealing. Soil Conservation Service recommendations call for thorough mixing of salt or polyphosphate with each 6-inch to 8-inch layer of soil to be sealed.

The effectiveness of compaction in reducing soil permeability is well known. Laboratory data in table 5 show that compacting chemically treated soil to 90 percent or more of maximum Standard Proctor density at about optimum moisture content results in permeabilities 1/100 to 1/1,000 of the permeability of treated soil at 85 percent of maximum standard density.

Field performance studies show results of the same magnitude: 100 percent success with compaction; no success without compaction.

Service recommendations call for compaction of each layer of chemical-soil admixture to a density of 90 percent of more of maximum Standard Proctor at or very near optimum moisture content.

Field experience indicates that two treated layers should be used when reservoir heads are more than 6 or 7 feet if minimum losses are to be obtained.

Table 5.--Relationships of compaction on permeability with TSPP and NaCl additives

Location	Soil Class[1]	Percent finer than--		Permeability				
		0.002 mm.	200--mesh	γ_d = 85 percent standard density no chemical	85 percent standard density + TSPP	90 percent standard density + TSPP	85 percent standard density + NaCl	90 percent standard density + NaCl
				Ft./day	Ft./day	Ft./day	Ft./day	Ft./day
New York	ML	16	50	0.1	0.001	0.00001		
Virginia	SC-SM	17	48	.137	.028	.0001		
Pennsylvania	SM-ML	14	50	.06	.004	.0002		
New Jersey	CL	28	80	1.19	.112	.0001		
Virginia	CL	45	90	.50	.136	.0001		
Virginia	CL-ML	20	90	.02	.001	.0003		
Virginia	CL	33	85	.01	.001	.00001		
Pennsylvania	CL	22	70	.05			0.01	0.0013
Texas	CL	30	70	.03			.005	.00001
Texas	CH	40	85	.14			.002	.0008
Texas	CL	27	55	.95			.002	.0001
Colorado	CL	32	75				.072	.004

[1] Symbols are those given in The Unified Soil Classification System, Tech. Memo. 3-357, Waterways Experiment Station, v. 1. 1953.

The relationship between compaction and successful sealing with dispersants involves features affecting permeability other than densification and reduction in pore space. The rolling or tamping action in itself promotes the formation of dispersed structure. The presence of the sodium chemicals during this compaction process hastens and improves the development of dispersed structure.

In addition to promoting dispersion, the compaction of the treated mixture results in more intimate contact of the dispersant with the soil particles. This reduces the possibility of losing some of the dispersant due to leaching when the treated area is inundated.

LABORATORY TESTING PROGRAM

Laboratory tests to determine the need for and kind of seal to reduce seepage losses include the standard index and classification tests (grain-size distribution by sieve and hydrometer down to percent finer than 0.002 mm., liquid limit, plastic limit, soluble salt content) and Standard Proctor compaction (Amer. Standards of Testing Materials:D698-58T).

Permeability tests are run in constant head permeameters at two or more void-ratios with and without varying applications of TSPP and/or NaCl. Permeability testing generally involves the following procedure:

1. One test on untreated soil compacted to 85 percent of maximum standard density at air-dry moisture content (dry tamp).

2. One test on untreated soil compacted to 90 percent of maximum standard density at optimum moisture content.

3. One or more tests with TSPP and NaCl or with NaCl compacted to 90 percent of maximum standard density at optimum moisture content.

Results of the air-dry or dry-tamp permeability test related to the percentage <0.002 serve as guides to the need for chemical dispersants on proposed projects and the probable amount of dispersant.

Untreated soils with more than 20 percent finer than 0.002 that have permeability rates of <0.001 ft./day, when compacted air-dry to densities not more than 85 percent of maximum standard, probably contain high amounts of exchangeable sodium, are naturally dispersed, and should have low natural permeabilities. Wet compaction of these materials without chemical additives should produce high impermeability.

Untreated soils with high percentages (40 percent or more) of clay-sized particles (<0.002 mm.) that show relatively high permeabilities (probably >0.05 ft./day) when tamped air-dry to densities not more than 85 percent maximum standard are probably well aggregated and may require higher than normal applications of chemical dispersants and more than 6 inches of thickness to produce satisfactory seals.

Results of the untreated and treated compacted permeability tests serve as indicators of the efficiency of the dispersant.

Soil Conservation Service recommendations consider the maximum permissible laboratory test permeability for effective sealing of most retention reservoirs as 0.001 ft./day at unit gradient. This permissible laboratory rate may be increased somewhat for projects involving short-time detention reservoirs such as overnight irrigation holding or equalizing reservoirs.

In addition to the above standard tests, a so-called "dispersion" test is routinely conducted as part of the index tests. This test measures the percentage of particles finer than 0.005 mm. in soil suspensions of same concentration as the hydrometer analysis, which contains no chemical-dispersing agents, and are subjected to minimum mechanical agitation (15 minutes soaking and air evacuation plus 30 end-over-end inversions of suspension in hydrometer jar). The ratio of 0.005 material in the nondispersed suspension to the 0.005 material determined by standard mechanical analysis is called the dispersion ratio.

For fine-grained soils, this test serves as indicator of (1) natural aggregation, (2) water stability of soil aggregates, (3) natural exchangeable sodium content. When interpreted with other index test values and dry-tamped permeability results, the dispersion test is useful in evaluating the need for and effectiveness of chemical dispersants.

FIELD INSTALLATION OF CHEMICAL RESERVOIR LININGS

Parts of the following writeup (the quoted material) on field procedure are taken from "Pond Sealing with Polyphosphates," R. F. Fonner, Engineering and Watershed Planning Unit, Soil Conservation Service, Upper Darby, Pa., (3). A number of ponds have been successfully treated in that area in the past few years.

1. Soil in the area to be treated should be moist to a depth of 10 to 12 inches. Desirable moisture should be near the optimum for maximum compaction. This will vary from about 12 percent for SM soil to 20 percent for CH soil.

   "Polyphosphates release water from soil and the job can easily become too wet to handle."

2. Clear the area to be treated of all vegetation and trash. Rock outcrops and other highly permeable areas should be covered with 2 to 3 feet of fine-grained soil.

   "In cavernous limestone areas, the success or failure of the seal may depend upon the thickness and compaction of this initial blanket."

3. Apply the dispersant (salt or polyphosphate) at prescribed rate with seeder, drill, fertilizer spreader, or by hand broadcasting.

   The dispersant should be finely granular with at least 95 percent passing No. 30 screen and $<$ 5 percent passing No. 100 screen.

4. Thoroughly mix the dispersant into the soil to a depth of 6 to 8 inches using a Seaman pulverizer, disc, or similar equipment. Some technicians insist that the soil should be pulverized before the dispersant is applied. Others (Pennsylvania) insist that the mixing of soil and dispersant be done by discing in two directions.

5. Compact the soil-dispersant mixture with sheep's-foot, rubber-tired roller, or wheel tractor to densities of 90 percent or more of maximum standard Proctor. In cases where soil is below optimum, sprinkle the treated area during the mixing operation.

6. If a seal blanket thicker than the normal 6-inch blanket is required, the seal should be built up by mixing and compacting each 6-inch lift.

7. The seal blanket should be protected from puncture by stock. The area near the high-water line should also be protected from erosion by covering with a 12- to 18-inch blanket of gravel or other soil material. Area where inflow is concentrated should be protected with riprap.

   Some areas of the country recommend mulching the entire treated area with straw to prevent cracking and gullying before the pond fills.

8. Whenever possible, flow into the treated pond should be controlled so that it takes several days to fill. This allows the reaction between soil and dispersant to progress slowly and prevent loss of dispersant by leaching. This is especially true when NaCl is used and compaction is not up to standards.

   "Water in a treated pond will stay muddy for many weeks because of the fine clay suspended as a result of the dispersing action of the chemical. The water will generally clear however."

COST OF CHEMICAL SEALS

Cost figures for installing chemical-seal blankets range from $300 to $3,000 per acre of surface treated. The cost for most of the small ponds (less than 1 acre) will generally run about $200 for chemical and $100 to $150 for labor and equipment.

EVALUATION OF FIELD RESULTS

The Soil Conservation Service has made many recommendations for sealing, and provided assistance in treating many small reservoirs. Unfortunately, there are very few data on performance of reservoirs after treatment.

We have many statements from all over the country citing the value of salt and polyphosphate treatments. We have a few citing the failure of these treatments. The information indicates that treatment by dispersants has reduced seepage losses by at least 50 percent in 90 percent of the sealing projects.

A few samples of statements regarding field performance from Texas in 1958 are as follows:

Blackland Resource Area

Deep, fat clay underlain by calcareous materials--

"Pond did not hold water before treatment. Treated with NaCl at 1 lb./3 sq. ft., dry disced into soil several times, no compaction. Results were good."

Edwards Plateau Resource Area

Deep silty clay derived from limestone--

"Pond treated with NaCl at 1 lb./sq. ft. Results were good."

Grand Prairie Resource Area

Deep, moderately permeable sandy clay and lean clay soils--

"Ponds treated in 1954 and 1955 with NaCl at 1 lb./3 sq. ft., distributed by hand and worked into soil with tandem disc--no other compaction."

"Mr. X states--Salt did it. I'd recommend it to anybody.

"Mr. A states--I know salt was the only thing that made mine hold. It certainly didn't before I used salt."

"Mr. B caught a rain the day after his pond was salted in 1954. His pond hasn't been dry since."

"Mr Y uncovered a rock ledge in his pond during construction. We discouraged use of salt in his pond but he felt that treatment was too cheap not to try. No evidence of holding water through the use of salt."

Rio Grande Plain Resource Area

Deep alluvial silts and lean clay soils--

"Mr. B's reservoir filled shortly after construction. In a short time, all water had seeped out. He applied NaCl at 1 lb./3 sq. ft. His procedure was to disc the reservoir and apply half the salt, then disc again and apply the remaining salt. Then he disced a third time. He filled the reservoir and it was still losing water at a noticeable rate. We recommended packing the bottom and sides with a sheepsfoot roller. He did this, and the reservoir is holding good."

A 1957 report from Pennsylvania describes a pond in Cumberland County which had been treated in 1954 with NaCl at a rate of 1 lb./3 sq. ft. in accordance with Soil Conservation Service recommendations. The salt treatment was not successful, due partly to freeze-thaw deterioration of the treated blanket. In 1955, the pond was retreated with tripolyphosphate at a rate of 1 lb./20 sq. ft. disked and compacted in the prescribed manner with blanket composed of two treated, compacted layers, each 6 inches thick. At the time of the report in 1957, the pond was and had been holding water within 14 inches of the spillway level.

Wickline (14) reports on two treated ponds in Greenbrier County, W. Va. One built in 1953 "leaked like a sieve." It would fill with each hard rain, but leaked out until about a foot of water remained. The pond was treated in 1957 with sodium tripolyphosphate at a rate of 1 lb./20 sq. ft., after which the pond level stayed at 6-foot depth, which was almost the exact elevation of treatment.

Another farmer shaped and treated a sinkhole with tripolyphosphate. Before treatment, the sink would collect 6 to 8 feet of water, which would seep out in a few hours. After treatment, the sink held 8 feet of water.

Johnson, Jacobson, and Schwab (5) report on two flood detention pools in the deep loess area of Iowa. These reservoir areas were treated with TSPP at rates of 1 lb./10 sq. ft. and 1 lb./20 sq. ft. with thorough mixing and compaction. Treatment reduced loss in waterhead about one-fifth.

The report of an evaluation study of pond sealing practices in Virginia (2) gives observations on treatment of ponds in five counties of the limestone area. This group concluded that all observed ponds treated with sodium polyphosphate could be classed as successful since they were all holding some water after a very dry summer and most of them had no water prior to treatment.

GENERAL RECOMMENDATIONS

General recommendations for the use of chemical dispersants to reduce seepage losses from small reservoirs are as follows:

1. Properties of soils generally impermeabilized by chemical dispersants:

   Soils should have 50 percent or more particles finer than 0.074-mm. diameter and 15 percent or more finer than 0.002-mm. diameter with less than 0.50-percent soluble salts (based on dry soil weight).

   Sodium polyphosphate effective on soils with plasticity index (P.I.) $>$ 5 and activity (P.I./percent $<$ 0.002) $>$ 0.30.

   Sodium chloride effective on soils with P.I. $>$10 and activity (P.I./percent $<$0.002) $>$ 0.6.

2. Character of dispersant:

   Tetrasodium pyrophosphate and trisodium polyphosphate preferred over other polyphosphate salts (commercial phosphatic fertilizer not acceptable).

   Chemicals should be finely granular with 95 percent passing No. 30 screen and less than 5 percent passing No. 100 screen.

3. Rate of application:

   Sodium polyphosphate = 0.05 to 0.10 lb./sq. ft.

   Sodium chloride = 0.20 to 0.33 lb./sq. ft.

4. Thickness of treated blanket:

   Finished blanket should be at least 6 inches thick. When reservoir head exceeds 7 feet and minimum seepage is desired, the blanket should be 12 inches thick, treated in two 6-inch lifts.

   There should be at least 2 feet of fine-grained soil over rock outcrops or other highly permeable materials in addition to the seal blanket.

5. Distribution of chemical:

   Area to be treated should be cleared of all vegetation and trash. Chemicals should be distributed evenly over the surface to be treated with drill, seeder, fertilizer spreader, or hand broadcasting. When broadcasting by hand, it is best to stake the area in small grids.

6. Mixing chemical with soil:

The chemicals should be thoroughly mixed into each 6- to 8-inch layer to be treated. Mixing can be done with disc, roto-tiller, pulverizer, or similar equipment. Disking in two directions produces best results. If moisture content of soil is not suitable for maximum compaction, water should be added by sprinkling during the mixing operation.

7. Compaction:

Compact each chemically treated layer to dry density of 90 percent or more of maximum Standard Proctor with soil near optimum moisture content. Results are best when moisture content is slightly higher than optimum.

Compaction can be done with sheepsfoot or rubber-tired rollers or wheel tractors. Sheepsfoot rollers are preferred.

8. Protection of treated area:

Treated areas should be protected from puncture by livestock trampling. Areas near the water line and where flows into the reservoir are concentrated should be protected from erosion.

Sediment-coagulating chemicals such as gypsum or iron sulfate should not be used to clear reservoir water which may be muddy after treatment.

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DISCUSSION

Wednesday morning - Second Session

Bower:

I would like to ask Mr. Decker if he has ever tried sodium carbonate rather than sodium chloride?

Decker:

Yes we have, and with about the same effectiveness as sodium chloride. There are places in the country where sodium carbonate is a byproduct which is cheaper in cost than some of these other salts. It does not work effectively, however, in the eastern kaolinitic materials. This is all tied up with this phosphate ion in these low-exchange clays.

Bower:

This is kind of amazing because we, from time to time, sodiumsaturate soils in great bulk, and we find that sodium bicarbonate or sodium carbonate is much more effective than sodium chloride. With sodium carbonate one obtains a practically quantitative absorption of the added sodium because one of the end-products is calcium carbonate, which precipitates and forces the reaction to the right. Whereas we can get 95 percent of the sodium in sodium carbonate absorbed by the exchange complex, we may only get 25 or 50 percent from sodium chloride.

Decker:

I don't know the reasons why, Dr. Bower. I do know that we have tried sodium carbonate in a place or two in Tennessee with some success and some failure. I am not sure why the reaction occurs; we also have tried sodium carbonate in some of the soils in New York where they have had very severe seepage problems--actually we have done very little work with sodium carbonate because salt is a more common source.

Bower:

Now the other thing I would suspect is that you get good results with sodium phosphate for the very same reason that I expect better results from sodium carbonate than from sodium chloride. Here you have insoluble calcium phosphates forming, which tend to force the reaction to the right, and a rather complete adsorption of the sodium. I would suspect that is why the sodium phosphates are highly effective rather than adsorption of phosphate. I don't quite see how phosphate adsorption induces dispersion.

Decker:

Well, this increases, actually, the net charge, which increases the number of water molecules that can go to hydrate the sodium or be oriented within the clay micelle to form a dispersed structure. There is a good deal of literature on that, Dr. Bower, by Lambe--for example--of MIT and Rosenquist. This is all appended in this article.

Lauritzen:

I would like to ask you: Do you have any information on the effect of these additives to scour, or leaching action; also, as compared to the untreated area?

Decker:

Well, they will erode faster, Cy, because they are in a dispersed condition and, of course, our recommendations would include some protection of this material by riprap or gravel blanket near the high-water line. I would like to ask Mr. Dirmeyer what depth of penetration he gets with his bentonite in water treatment on various medium-grain, sandy soils?

Dirmeyer:

We have done some work in the laboratory on what you might call a groutability index. We have also looked into the penetration question from the standpoint of filter bed theory. As yet, we are not ready to make a definite statement regarding depth of penetration of bentonite versus a size description of the canal bed material, such as, for example, its median diameter or its D_{50} size. In fact, I suspect that it is not that simple; for example, a sodium montmorillonite can bridge voids much larger than one might suppose from the effective size alone of its dispersed particles. This seems to relate to edge-to-edge lineup of montmorillonite particles in the filter cake. In general, it seems as though the penetration of bentonite into a medium-grained sandy soil, as you mentioned, would relate to the amount of fines (minus 200 size) in the sandy soil. Any such soil with over about 5 percent fines probably will not be penetrated by dispersed bentonite. In any case, because of the extreme variability of field soils--even those described as medium-grained sandy soils--the degree of penetration of bentonite will also be quite variable, ranging from a surface seal only to a penetration in excess of several inches.

Decker:

What I am wondering is, could you tie, for example, penetration to percent porosity for a medium-grain size, or some other feature?

Rollins:

I have a little information that might help you. We just completed a field study where we compared several different methods of applying bentonite in a fine sand. The waterborne material penetrated the sand down to 2 inches in quantities large enough to measure, but most of it was in the top inch.

Tovey:

I have a question I would like to direct to Dr. Lauritzen pertaining to canal linings. About a year or so ago you were doing work on an improved slip form. Can you tell us a little about that? What progress you made?

Lauritzen:

This work is being done primarily by Frank Haas in my office and is reported in Farm and Home Science, September 1962.

SESSION 4.--CHAIRMAN, LLOYD E. MYERS

PROBLEMS IN SEEPAGE EVALUATION AND CONTROL[1]

C. C. Warnick[2]

INTRODUCTION

Seepage evaluation, like all the measurements men have tried to make, tends toward the need and desirability of a better and a more precise quantitative value. Fortunately, today the tools are better, but really we are still very crude in our methodology. After seeking better ways, it is discouraging to report we do not have an answer sufficiently accurate to indicate where we should control seepage or where it is the problem.

HYPOTHETICAL MEASUREMENT DESIRED

For purposes of optimum usefulness, an idealized type of measurement might be referred to as one which will give an integrated value of rate loss over a length of channel 500 to 1,000 feet, with an accuracy of plus or minus 0.1 cu. ft. per sq. ft. per day. While dreaming, let us put another restriction--that the measurement takes no longer than a day to make and compute the results. There is no proof of these hypothetical values. They are just gleaned from experience of trying to measure and to evaluate losses from ordinary canals. This does not cover the need to know precise paths of seepage movement that may be causing damage at particular points.

REALISTIC AND PRACTICAL MEASUREMENTS

Starting with a useful measure of seepage evaluation, consider the growth of vegetation adjacent to the canal. This can certainly tell us in a qualitative way whether loss is serious, but it is only useful to an experienced person in a given locality, and it may take years to train a person to recognize the seriousness of loss. In Arizona an Idahoan would only be guessing if he read the signs of the vegetation. To the irrigation district manager, it is good counsel to say, use this with discretion with your maintenance forces.

Fluctuation of the water table in auger holes or special observation wells can often tell the changes in loss and general gradient of seepage movement. It can give no quantitative value, but with changes in operation in the canal, empty or full discharge can point up qualitatively a problem area.

There have been many attempts to use current meters or other channel discharge-measuring systems but such measurements have been very restricted in usage.

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[1] Contribution from the University of Idaho, Moscow, Idaho.

[2] Associate director, Engineering Experiment Station.

For an inflow-outflow test of a 500-foot length of channel having a 20-foot wetted perimeter and a loss rate of 0.1 cu. ft./sq. ft./day, it would mean that only 0.001 c.f.s. would be lost in that section. So if a channel of that size were carrying a normal flow of about 50 c.f.s., the futility of trying to make direct inflow-outflow measurement of the loss is noted, because a 3 percent error might be read with a current meter amounting to 1.5 c.f.s. Hence, only very long sections should ever use an inflow-outflow test.

Ponding tests have come to be somewhat of an accepted standard for measuring seepage in canals. But the experience of the University of Idaho in a specially prepared section should illustrate a caution. Four sections of a special bypass canal were isolated for study in 1961. Each channel was 200 feet long, separated by a watertight bulkhead. This channel was parallel to an existing channel, but far enough away to be free of the effects of the regular canal. It was hoped that the channel represented reasonable uniformity. Soil profiles of a line through the area showed some variation, but certainly not more than one would expect in many miles of canal of the Black Canyon Irrigation District. This was tested from 10 July to 13 September 1961. There were 41 separate days that independent drops in water surface were observed. At all times the channel elevation was not allowed to drop more than 6 inches, and when not under independent testing, the water level was kept at the same elevation in all ponds. The mean seepage loss for the four channels varied from a high of 1.09 cu. ft./sq. ft./day to a low of 0.41 cu. ft./sq. ft./day. Individual deviation of loss rate from the mean of the four ponds was as great as 0.65 cu. ft./sq. ft./day, but averaged 0.33 cu. ft./sq. ft./day. The average deviation for the 41 days of testing indicated on the average that there could be a 38 percent variation from the mean of all tests. This variation in ponding measurements in the four ponds is shown graphically in figure 1. Naturally there were many factors that made these inconsistencies. Care was taken to minimize the human element in measuring, but it was not isolated as a contributing factor.

Certainly the variation was somewhat seasonal with a generally downward trend early in September. Two ponds showed very marked reduction during early September. A

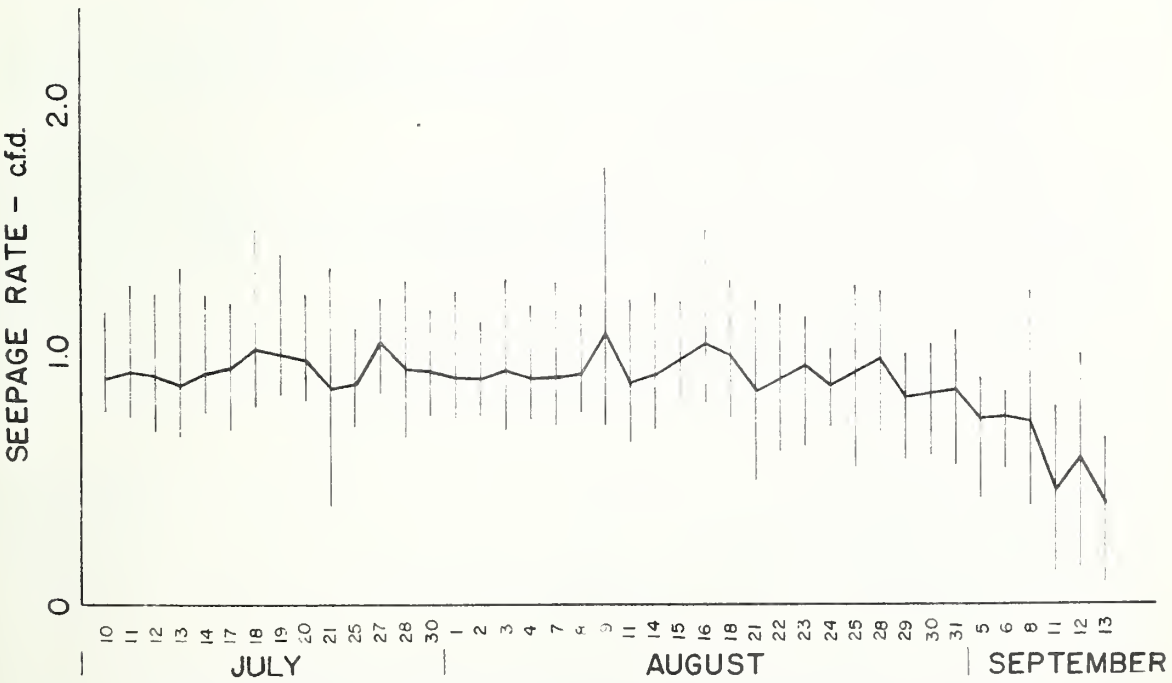

Figure 1.--Variation in seepage ponding rates in four separate but adjacent ponds.

suggestion of relation was noted in the fluctuation in seepage rates and the fluctuation in barometric pressure. The temperature variations did not seem to influence loss variations as much as one would suspect.

Seepage meters were tested in the same channels. Individual measurements in a particular pond varied widely from a maximum of 680 percent of the median ponding rate to a minimum of 21 percent. Five successive days of ponding tests in Pond No. 1, during which 26 separate seepage meter measurements were made, showed the median ponding loss rate under a unit head calculation to be 0.53 cu. ft./sq. ft./day and a median rate of loss indicated by the seepage meters was 0.65 cu. ft./sq. ft./day. A graphical presentation of this comparison is shown in figure 2. This was a very reasonable agreement, but it should be noted that it took 5 days of effort to obtain the seepage meter data. A single measurement could have been off by the 680 percent.

PROBLEMS OF VARIATION

A very challenging problem when making a seepage test is the effect of silt and bedload on the seepage rate. In experimenting with seepage rings it was found that a day or two of the accumulation of silt from ordinary canal water can change the loss rate by as much as five times. The flow can change to unsaturated flow immediately below the soil surface, thus changing the environment for the effects of biological activity.

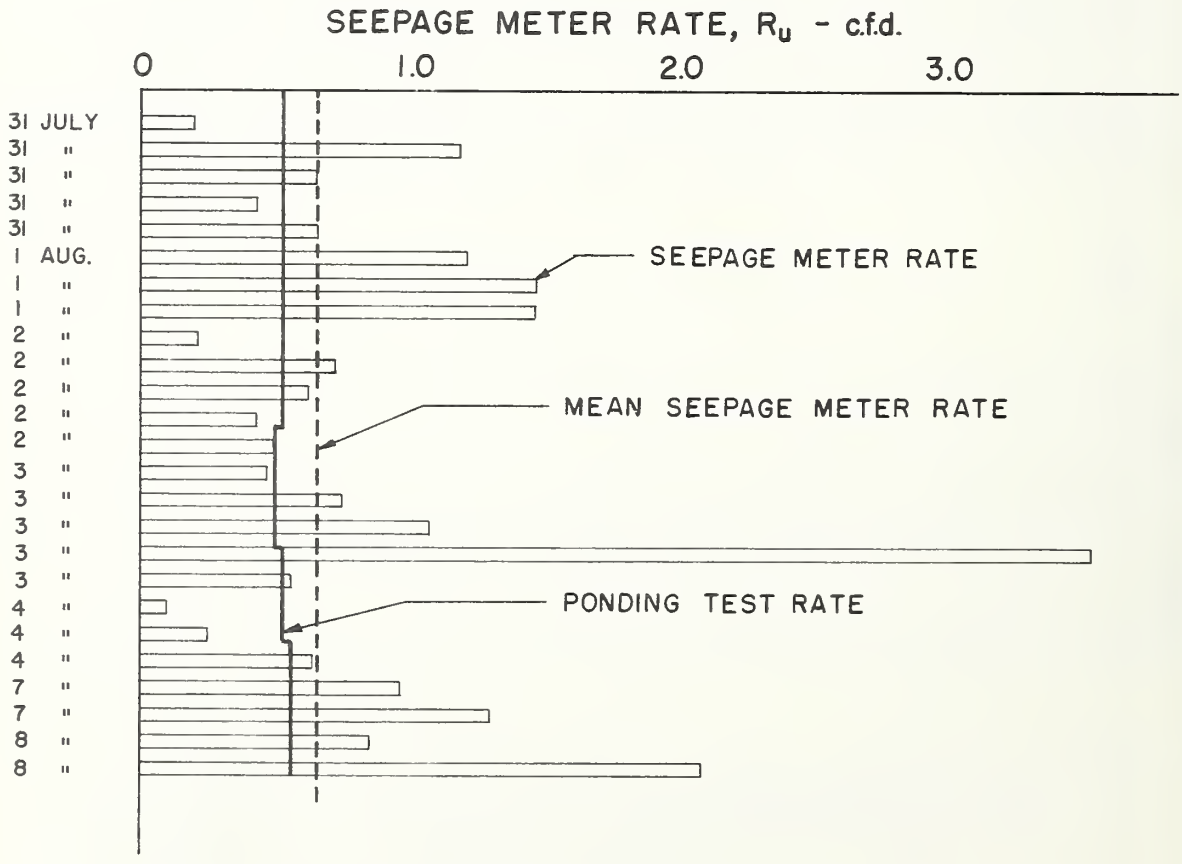

Figure 2.--Comparison of seepage rates as measured by the seepage meter and the ponding method.

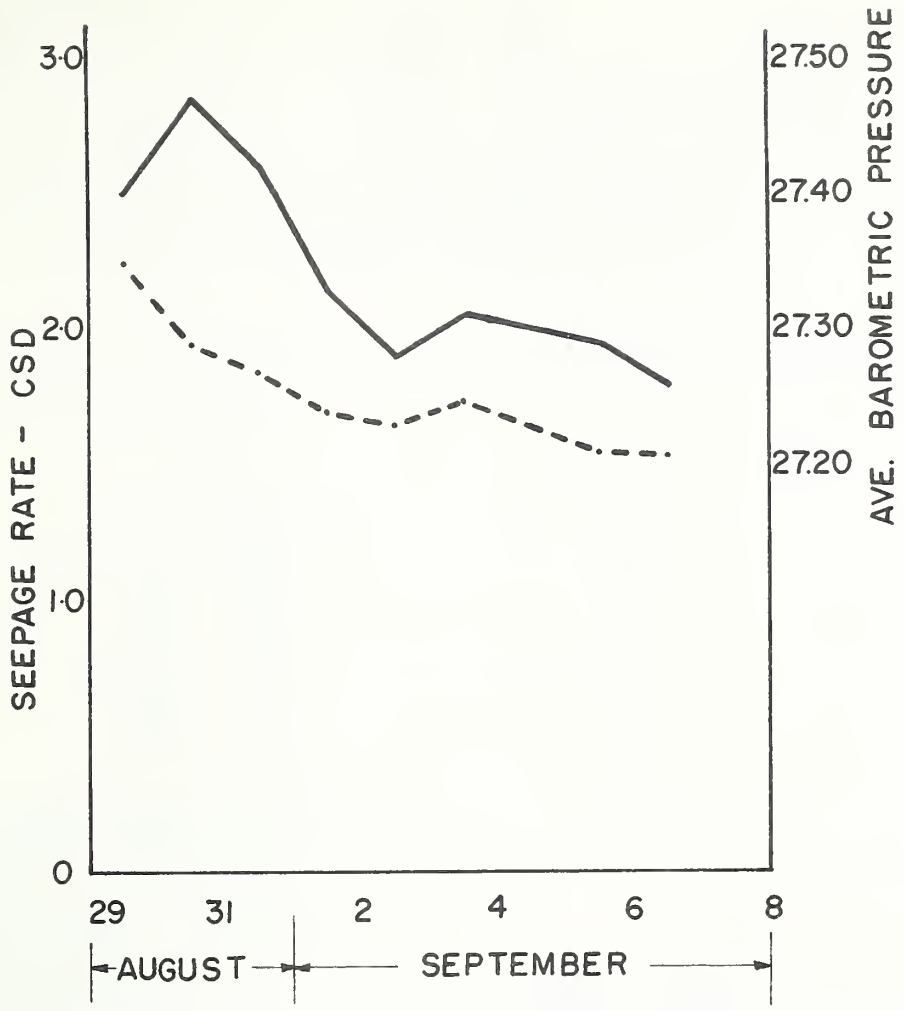

Figure 3.--Indicated fluctuations in seepage loss rates as affected by barometric pressure.

It has been suggested that fluctuations in barometric pressure may cause variation in seepage loss rates. Experience with tests conducted in a ponding test this past fall (1962) tends to illustrate this possible relationship. This is shown in figure 3. At first thought it seems that a decrease in barometric pressure of tenths of inches of mercury should have little effect on seepage rate, but this pressure change may directly change the size of air voids in both the porous media through which seepage water is moving as well as the air voids in the water itself. The magnitude of the influence of barometric pressure is not known, and experience indicates that in different ponds the influence may vary in magnitude.

To illustrate the effect of human error, a brief test was made on the way in which a seepage meter was placed in the channel. Pushing the seepage bell into the bottom of the channel in series of tests at one location showed the loss in the meter to be 123 percent of the loss as measured by an independent means of ponding. Gently stepping on the meter to position the meter did not change this relationship, but hammering on the seepage bell increased the meter rate over the control ponding rate to 158 percent. Obviously the minimum disturbance of the seepage meter is the best practice. Two types of devices for controlling the head on water flowing from the seepage meter are illustrated in figures 4 and 5. Preference is given to the use of the seepage meter with a Mariotte siphon.

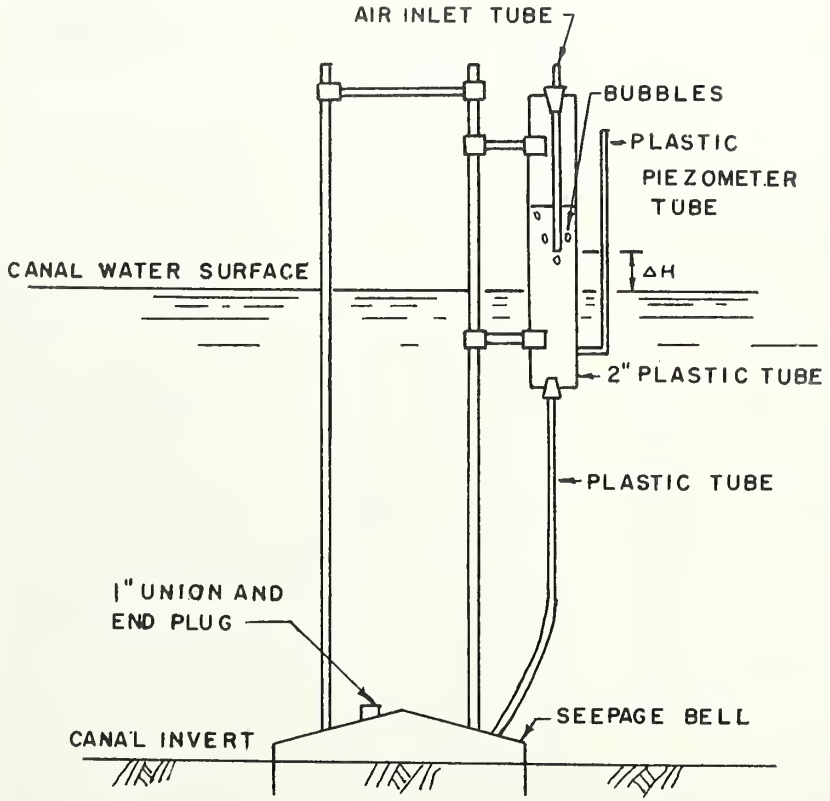

Figure 4.. Schematic of Mariotte siphon reservoir used to impose a constant head on the seepage meter bell.

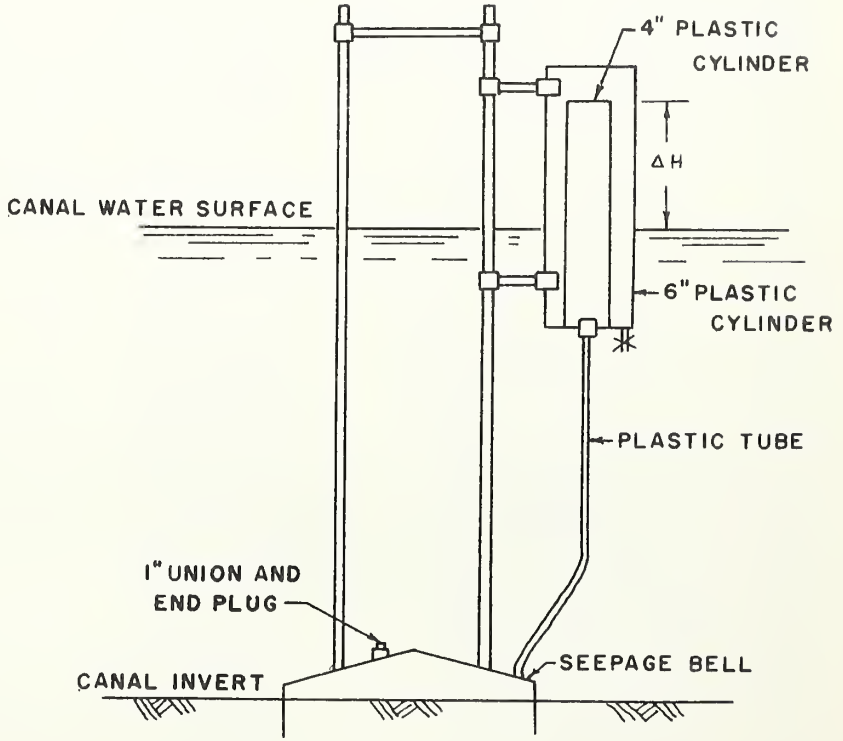

Figure 5.. Schematic of constant head device used to impose a constant head on the seepage meter bell.

Work done at the University of Idaho with well point piezometers has shown that the permeability of the saturated flow zone below a canal can be determined by observing the rate of drop down of the water surface in the piezometer. This is discussed in a paper prepared by the writer and D. W. Hendricks now being considered for publication by the Irrigation and Drainage Division of the American Society of Civil Engineers. A problem here is that any quantitative evaluation of seepage will be based on a point measurement of permeability and an evaluation of the hydraulic gradient at a point. Experience with measuring permeability indicates the accuracy may be in the range of 1 to 10 feet per day, which certainly leaves much to be desired in giving a good quantitative figure.

All these methods have their uses and must be resorted to in some instances. Research to find a more refined method is certainly a needed area of endeavor, but caution on hope for spectacular success is given.

CATIONIC CHEMISTRY IN WATER CONSERVATION[1]

Jack N. Dybalski[2]

The primary objective of this paper is to introduce and discuss the concept of cationic surface chemistry and its relationship to the general field of water conservation. To help clarify the introduction of this concept, however, I will first define and describe the phenomenon of surface activity and the three main types of surface-active chemicals or surfactants which are in use today.

Certain substances, even when present in very low concentrations, possess the unique property of altering the surface energy of their solvents to an extreme degree. Almost always, a lowering rather than an increase of the surface energy is effected. Substances or solutes possessing such properties are known as surface-active agents or surfactants and their unique effect is known as surface activity.

By broad definition then, surface-active chemicals are soluble substances whose presence in solution markedly changes the properties of the solvent and the surfaces they contact. They are categorized according to the manner in which they dissociate or ionize in water and are characterized, structurally, by possessing a molecular balance of a long hydrophobic, hydrocarbon "tail" and a polar, hydrophilic "head."

Surface-active agents owe their physicochemical behavior to their property of being adsorbed at the interface between liquids and gases or liquid and solid phases. They tend to concentrate in an oriented manner, at the interface, in such a way that, almost entirely, they turn a majority of their hydrophilic groups toward the more polar phase and a majority of their hydrophobic groups away from the more polar phase and perhaps even into a nonpolar medium. The surface active molecule or ion, in a sense, acts as sort of a bridge between two phases and makes any transition between them less abrupt.

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[1]Contribution from Armour Industrial Chemical Co., McCook, Ill.
[2]Section leader, Applied Research Laboratory.

Basically there are three types of chemical surface-active agents which are classified according to their dissociation characteristics in water. These are:

1. Anionic surfactants--where the electrovalent and polar hydrocarbon group is part of the negatively charged ion, when the compound ionizes:

ANIONIC

$$CH_3(CH_2)_{16}\ COO^-Na^+$$

2. Nonionic surfactants--where the hydrophilic group is covalent and polar and which dissolves without ionization:

NONIONIC

$$CH_3(CH_2)_{16}\ COO(CH_2CH_2O)xH$$

3. Cationic surfactants--where the electrovalent and polar hydrocarbon group is part of the positively charged ion when the compound ionizes.

CATIONIC

$$CH_3(CH_2)_{17}NH_3^+\ CL^-$$

It is readily apparent that differences in the physical molecular structure of the various surfactants is slight. If these slight structural differences were the only factor with which the surfactant types gained distinction, they would be unimportant. What is important, however, is their electrochemical properties and their behavioristic relationship to surfaces which distinguishes one from the other. Physical chemists, who are in the midst of compiling data to explain the action of surface-active agents on various liquid-gas and liquid-solid interfaces, have reported that due to higher kinetic energy forces and larger electrophoretic mobility values for cationic surfactants than for the anionic or nonionic types, the cationic surfactants possess an extremely high degree of affinity for almost all surfaces. By virtue of their higher free energy, the cationics possess the ability to migrate to, be adsorbed by, and be electrochemically bonded to the negatively charged surfaces which predominate in our atmosphere and environment. The affinity of a cation for mineral surfaces, for example, is stronger than its affinity for water and will displace water from those surfaces irreversibly.

These characteristics, observed and investigated through less than a score of years of research, have led to widespread industrial use of a group of organic nitrogeneous chemicals which are derived from fats. The availability of a wide range of these cationic chemicals--each with distinctive solubility, reactive and substantive characteristics--has allowed researchers to be quite selective in their search for ideal properties.

The ability of these cationic agents to be attracted to textile fibers from solution has led to almost worldwide use as textile softeners and conditioners. They have been found to have utility in fuel oil as sludge inhibitors, in gasoline as deicers, in paint as pigment

dispersants, in ore reclamation as selective flotation agents, in dyes as levelers, in rubber for mold release, and in plastics as antistats. They react with clay to make it waterproof and with herbicides to form products with greater surface adherence and water resistance. The road construction industry has found them to be effective in bonding asphalt to water-wet mineral aggregates and to stabilize dimensionally poor soils.

Although many more applications could be related to illustrate the inherent versatility of the cationic agent and its adsorptive property, it is the ability of selected cationic agents to function as emulsifiers for various oleoresinous substances which has led to the recent development of materials proven potentially useful for water conservation.

Cationic asphalt and petroleum resin emulsions are rather recent innovations which, in many road construction techniques, are rapidly replacing the more conventional anionic asphalt emulsions, the solvent cutback asphalts, and, in some instances, even the hot asphaltic concrete mixes. The ability of the cationic emulsifier to transmit its electrochemical character to the dispersed bituminous droplets has resulted in the development of construction materials which function effectively even under the most unfavorable climatic conditions. The deposition of a cationic emulsion onto a surface is primarily an electrochemical phenomenon which, due to the inherent affinity or substantivity of the cationic agent, begins to take place at the moment of contact. Surface and atmospheric conditions have little effect, if any, on the adherent properties of the cationically treated asphalt, and the presence of moisture on a surface is no deterrent to adhesion. The strong preferential attraction of the oriented cationic agent actually displaces water from the surface, thereby creating a common chemical bond between it and the asphalt. The bond is such that an antistripping effect is obtained.

It was a laboratory observation of the ability of cationically emulsified asphalt droplets to retain their strong migratory and adsorptive properties even when extensively diluted in water, which led to the evaluation of specifically formulated emulsions as waterborne soil sealants. Our present formulations have, as a result of considerable laboratory and field evaluations, progressed to the point where they have some practical utility. Our initial formulations, first evaluated about 2 years ago, proved to be sensitive to dissolved anions and cold water. The incompatibility of some cationic agents with the relatively large concentrations of dissolved anions, such as carbonate and bicarbonate, found in much of the water in the southwest, was eventually overcome by formulation modification. The effect of cold water in reducing the rate of migration and strength of adsorption has also been overcome by increasing the overall kinetic and latent thermal energy of the emulsion system.

The present emulsion systems, although still not optimum, appear to be quite effective and lasting for static water systems. A typical result obtained with 1/2 gallon of a specific formulation per square yard of wetted soil shows a reduction of seepage from 6.48 in./hr. to 0.66 in./hr. after a 48-hour period. Using 1 gallon per square yard of soil provides a reduction from 7.68 in./hr. to 0.31 in./hr.[3] Material treatment costs for these treatments would range between 12 and 30 cents per square yard of wetted soil.

While penetration of several inches of asphalt can be obtained in granular soils of fairly even gradation, the results quoted were obtained in a fairly silty soil with only 1/4 to 1/2 inch of surface film. Attempts have been made to formulate true, cationic emulsions with better penetration characteristics for finely divided soils, but this is like fighting the inherent nature of the beast. It is possible to formulate asphalt emulsions which

[3] Soil and Water Conservation Research Division, ARS, Tempe, Ariz.

will more effectively penetrate these soils by reducing the cationicity of the system toward a more nonionic type. However, this only serves to reduce the adherent properties and results in a layer of friable asphalt beneath the surface of the soil.

We consider the surface films which were obtained using a ponding technique, to be satisfactory for static water. Some resistance by these films to low rates of flowing water has also been obtained, but this is an area where the film obtained is only as strong as the petroleum base used in the emulsion. While recent investigations have disclosed other petroleum bases which, when deposited from water, have substantially greater flow resistance, only additional research and field evaluations will tell whether optimum formulations are feasible for flowing water as well.

Construction of low-cost water harvest areas is another application where the bonding characteristics of asphalt deposited from cationic emulsions have been used to advantage.

These emulsions have been evaluated simply by spraying the emulsion on herbicidally treated soil and measuring the water collection effectiveness and the life expectancy of the asphalt film. Applying enough emulsion to provide 0.1 lb. of asphalt per square foot of soil, for instance, produces a soil cover that is highly impermeable. When desired, additional water resistance can be quickly obtained by spraying the asphalt surface with a very dilute solution of an additional cationic agent. The materials cost of the combined herbicide, emulsion, and water-repellent treatment on a 100-sq.-ft. plot is approximately 10 cents per square yard. This type of installation, after 1 year in service, still collected ten times as much water as the untreated section. These thin films of cationically treated asphalt apparently also possess unusual weather resistance. Some test panels have shown no deterioration after two years of exposure to the Phoenix climate.

The 0.1 lb. of asphalt per square foot used here, however, appears to be the minimum it is possible to use effectively. Water collection and weatherability can, of course, be improved by using more asphalt or by using the cationically deposited asphalt as an adhesive for more sophisticated films. Polyethylene, aluminum, a fiberglass-aluminum sandwich, and butyl rubber can be effectively and durably held to the soil. Of course, the materials cost goes up significantly when materials of this nature are used.[3]

Obviously, then, each person interested in installations of this type will have to weigh the cost of materials and installation versus the durability which can be expected and versus the amount of water which can be potentially accumulated.

There are still a few other ways that cationic emulsions can be used and these are primarily in the area of maintenance and construction of hydraulic structures.

1. These emulsions can be topically sprayed on old concrete and bituminous structures to generally seal and rejuvenate worn surfaces.
2. The emulsions can be combined with certain hydrocarbons and aromatic solvents to obtain penetrability into dry soils. These mixtures can be used for construction of water harvest areas and for the topical stabilization of loose, unvegetated soil as an erosion control measure.[3]
3. The emulsions are also excellent mulching materials which enhance the germination of cover crops by increasing the heat and water retention capabilities of soil.

[3] Soil and Water Conservation Research Division, ARS, Tempe, Ariz.

4. Herbicide-emulsion combinations have been applied and found to be considerably more effective than the herbicide by itself. The deposited asphalt actually tends to fix the herbicide to the area being treated and provides more resistance to water.
5. Certain formulations are also quite effective for stabilizing soil. Sandy, silty, and clayey soils have been made more resistant to water absorption, have had their load-bearing capacity increased and have been generally dimensionally stabilized. Because of the combined water resistance imparted by both the asphalt and the cationic agent, it also appears to be possible to completely impermeabilize soil to a depth of 2 to 18 inches.
6. A rather recent technique, which is being investigated for the applications of primary canal linings and for the rejuvenation and sealing of old linings, is the utilization of loose, free-flowing mixtures of sand and other fillers with specific cationic emulsions. These mixtures, called slurries, were originally developed for road maintenance as a resurfacing for all types of pavements and for secondary road construction as a bituminous wearing surface over stabilized bases.
7. The advantage of the cationic slurry is that structurally well designed and impermeable layers can be applied and used in a very short period of time.
8. For canal work, we have found that a pneumatic device capable of shooting these wet mixes is one technique which appears to have practical merit. However, this technique is still in a stage of development with considerably more field experience necessary before all requirements can be satisfied.

It is hoped that this paper has served the purpose of furthering the general knowledge of cationic surface-active agents, their versatility and their behavior. I have attempted to disclose their adsorptive and bonding characteristics and how these properties relate to water conservation techniques.

Admittedly and necessarily, I have had to cover many areas of application without being too specific in any of them. However, if any one of the applications discussed is of particular interest, I will be happy to provide more specific data where available.

It is our intention, of course, to continue our cooperative efforts with people in this field and to continue our own work on the refinement and development of cationically based materials which have practical value for water conservation.

A POLYMER COMPOUND FOR SEEPAGE REDUCTION [1]

J. Harlan Glenn [2]

The Imperial Irrigation District and the Coachella Valley County Water District realized in 1955 that future demands for water in the western part of the United States would require that the water being irretrievably lost by seepage be conserved by some manner not then generally practiced.

[1] Contribution from the Brown Mud Co., Torrance, Calif.
[2] Chief engineer.

The two districts have a major problem because the All American Canal system is in use the year round, and a shutdown period long enough to construct conventional linings would be impossible. The normal alternative would be construct a new parallel canal at an extremely high cost. A method of reducing the seepage loss by the addition of some material to the water under operating conditions seemed highly desirable. First thoughts on correcting the seepage losses centered around the use of a sedimenting procedure with either bentonite or local clays, and an experimental program was started.

Brown Mud Company became interested in this program because of their extensive experience in the mixing and handling of clay-laden slurries under adverse conditions. A survey of the canal system conditions and problems was made, and it was determined that a sedimentation program using bentonite or local clays would result in a very temporary reduction in seepage and that some other method of economical control of seepage should be investigated. This was a rather startling conclusion since clay has been successfully used for reducing or stopping seepage for centuries in many localities. In the Coachella Canal, both the chemical environment and method of placement to be employed were against the successful completion of the proposed experiment. The success of clay treatments has hinged on the ability of clay particles to swell when wetted so that the voids of a given soil are filled with the "swelled" clay. This swelling is associated with the adsorption of water on the surface of the clay particles to form an oriented water envelope around the particle, or a nonliquid water layer. Adsorbed inorganic cationic components and various types of ionic organic materials control the nature and extent of oriented water development, with the inorganic cations probably being the most important. If the adsorbed cation is predominately calcium, an abundance of water tends to develop very well oriented water to a thickness of about four molecular layers with any additional water not oriented. There is a sharp break between liquid and nonliquid water. In an air-dried state, calcium ions tend to develop two molecular layers of well-oriented water. In general, magnesium ions follow the calcium ion pattern except that the thickness of well-oriented water is probably slightly less in both the saturated and air-dry state. The sodium ions, on the other hand, favor the development of very thick layers, probably tens of molecular layers, if the water is available. The oriented water envelope seems less rigid than that associated with calcium and magnesium ions and there is no abrupt separation between oriented and liquid water. Sodium ions favor the development of a single molecular layer in an air-dried state which results in a tremendous difference between the thickness of adsorbed water in an air-dried state and the potential thickness if moisture is available. It follows that sodium clays have very high swelling potential and liquid limit values.

Untreated clays or soils usually contain a mixture of exchangeable cations, and the gradual replacement of one cation by another may have very little influence on adsorbed water development up to a certain critical point at which there is a very abrupt change in the stable thickness. When sodium ions are being replaced by calcium or magnesium ions, there is a distinct change in volume, and subsequently an increase in permeability of a soil and possibly a breakdown in the continuity of a layer composed mainly of clay particles.

It follows from this analysis that both the type of operation with respect to wet-dry cycles and the chemical constituents of the water conveyed in an irrigation system should be carefully considered in an earth-lined canal or reservoir. The All American Canal system is operated 365 days each year so wet-dry cycles and cracking due to air drying of clays would not be a problem. The Colorado River water conveyed in the system has a very high concentration of calcium and magnesium salts (averaging about 7.9 meq./liter total, or a hardness of about 400 p.p.m. as calcium carbonate) and therefore is of major concern when earth linings or sedimenting procedures are contemplated. The effect of a particular water on a clay to be used in a sedimenting procedure can be tested by means of a very

simple laboratory experiment. A slurry of the proposed sedimenting material is prepared as it would be in the field, including any dispersants or flocculants that would be used. This slurry is allowed to settle in a laboratory filter cell for a given length of time or long enough to build up the desired sediment layer thickness. The remaining slurry is then decanted off and replaced by the normal canal water, and loss rates are determined over a period of time. If the loss rate increases rapidly, the clay material is not suitable as a sediment lining where that particular canal water is used. If the loss rate decreases consistently, it is an indication that the clay itself is suitable for sedimentation, although other tests are still required to evaluate its overall effectiveness. The results of such a test on a bentonite sediment with Colorado River water is shown in figure 1. As a control, the result of a bentonite sediment used with a sodium-rich soft water is also shown. Sediments taken from the bottom of the Coachella Canal were treated with dilute caustic soda to return the ion exchangeable fractions from the calcium and magnesium form of clay to a sodium-type clay and the sedimentation test was made. The results show why the sediments already present in the canal are not controlling seepage. Sedimentation cannot possibly work in this system unless it is carried on continuously by a supply of naturally silty water. Bureau of Reclamation published data clearly show that seepage increases when clear water follows a silty water flow.

Soil Sealer-13 (SS-13) was developed specifically for the situation where the presence of calcium and magnesium environment has minimized the effect of clays naturally present in the soils with respect to seepage reduction. In general, SS-13 consists of the sodium salts of polymerized organic molecules capable of adsorption on the surfaces of both clay particles and silaceous materials when applied in very dilute form. It has been established through the testing program that a 1,000 p.p.m. suspension of SS-13 in contact with a soil bed for 16 to 48 hours provides an optimum economical treatment in most cases. The sodium, calcium, and magnesium salts are all surface active so that they can become attached in any normal environment. The calcium and magnesium salts of the polymers are almost completely insoluble in water, so the polymers are added in emulsion form to prevent premature reaction in hard water. Once adsorbed on the surfaces of clay particles and silaceous material, the large polymer molecules extending into the void space increase the resistance to water flow through the soil by trapping or holding the water in place.

The first application of SS-13 was made in a 7.8-mile section of the Coachella Canal on October 29, 1957. A concentration of 440 p.p.m. was used in this test. It was planned to use inflow-outflow measurements in this short reach as a means of evaluation. This method of measuring seepage proved impractical, as you have probably already inferred from the papers previously presented on seepage evaluation. Operational records over a 5-year period have enabled us to evaluate this treatment's effectiveness only in general terms.

The first well-controlled test was made here in Phoenix on the South Canal under the sponsorship of the Salt River Project and Region 3 of the Bureau of Reclamation. This canal had been gunited in the 1920's and had badly cracked and spalled by the time this test was started in 1958. The loss rate before treatment with SS-13 in this section was about 1.0 cu. ft./sq. ft. wetted area/day. The results of this test are well documented. The results for the first 2 years following treatment were presented by Paul E. Ruff, School of Engineering, Arizona State University in his report for the Salt River Project and Bureau of Reclamation, and are shown in figures 2 and 3. The loss rate at the end of 3 years following treatment was essentially the same as the rate at the end of 2 years. Figure 3 presents some very useful information for computing the cost of conserving water being lost by seepage. The South Canal can be treated with SS-13 for a cost of $4,238 per mile (actual figure submitted in contract negotiations), and would save 4,194 acre-feet of water in 2 years at a cost of $1.01 per acre-foot for the 2-year period shown.

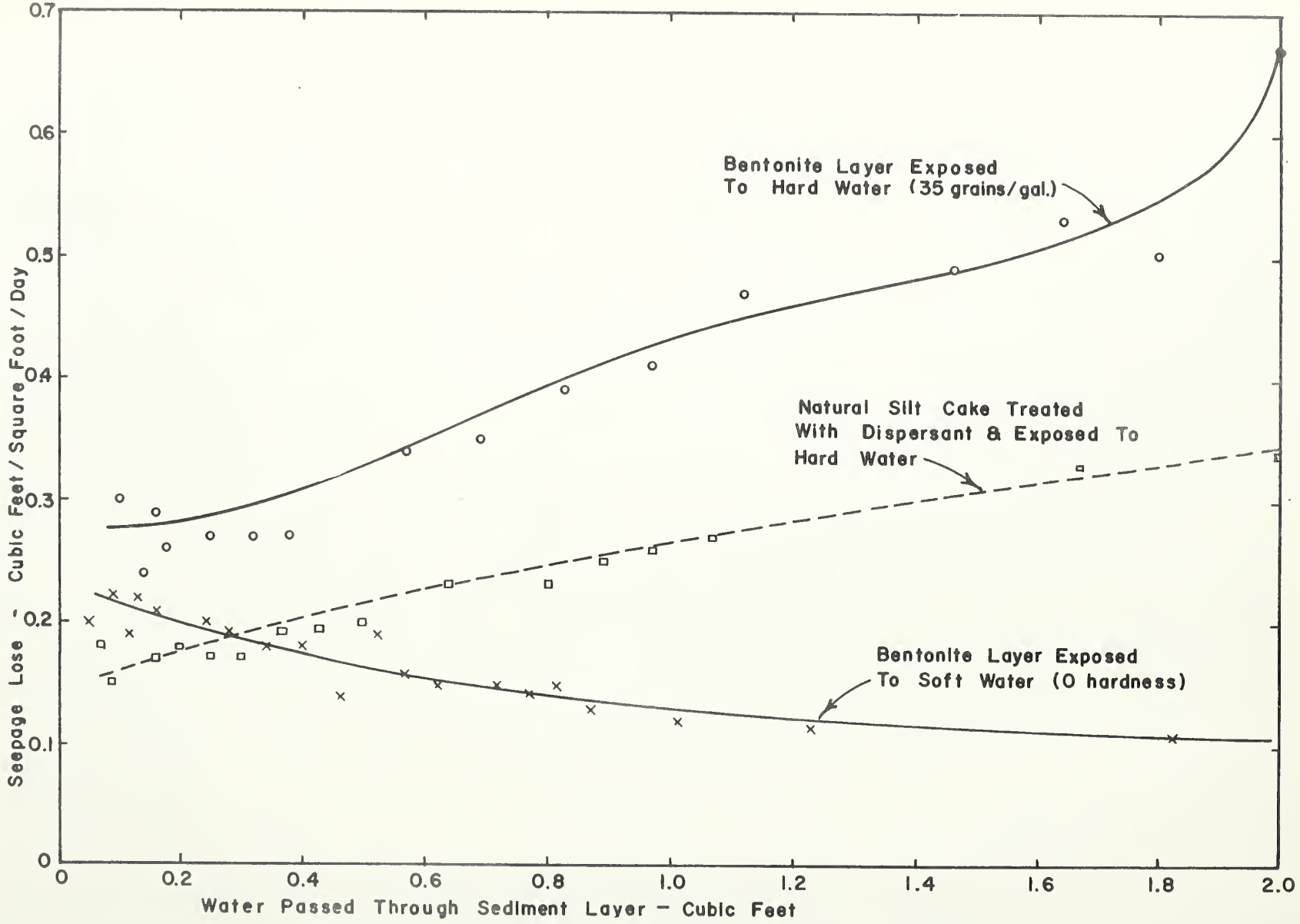


Figure 1.--Effect of hard and soft water on sediment seals.

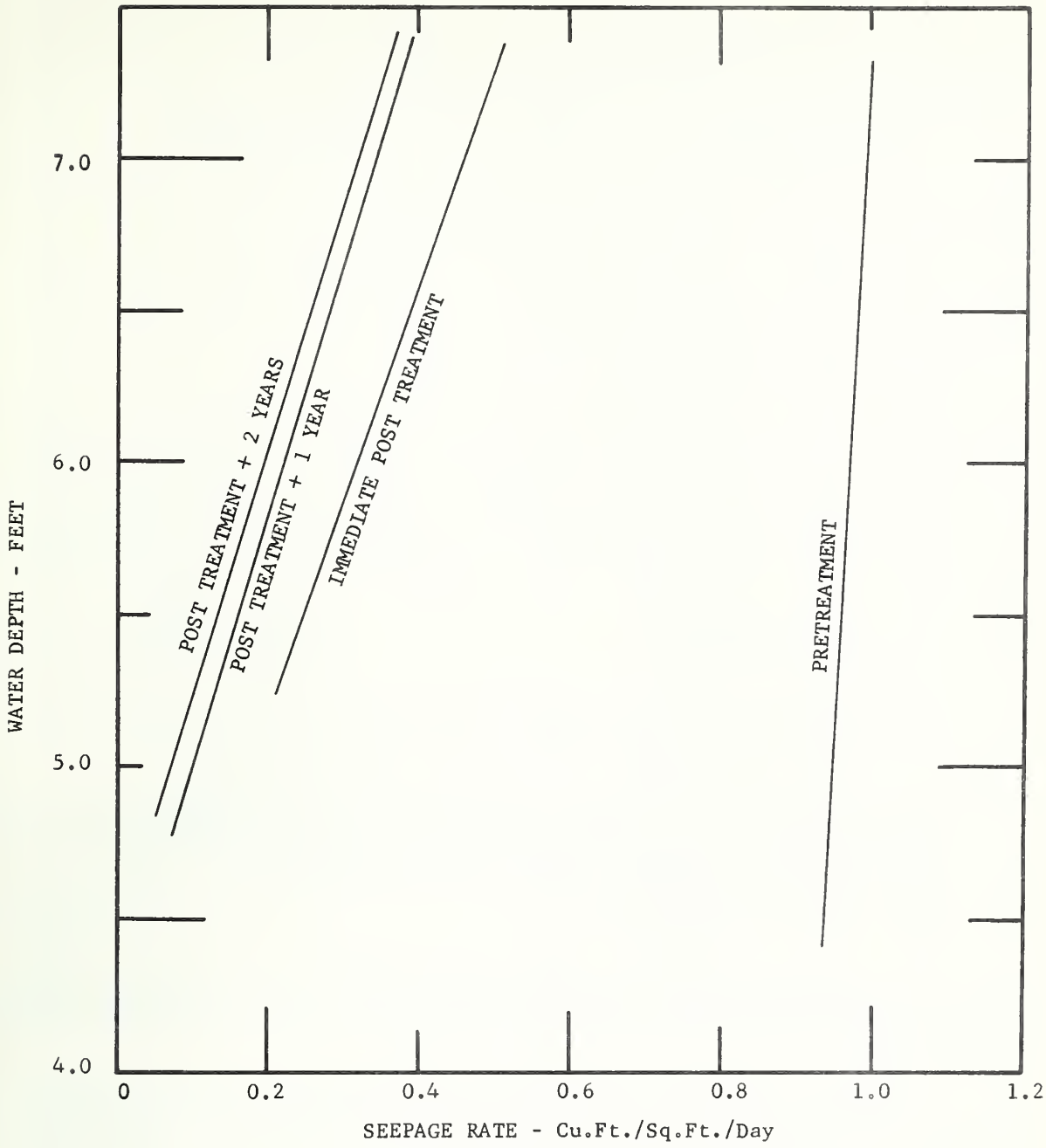
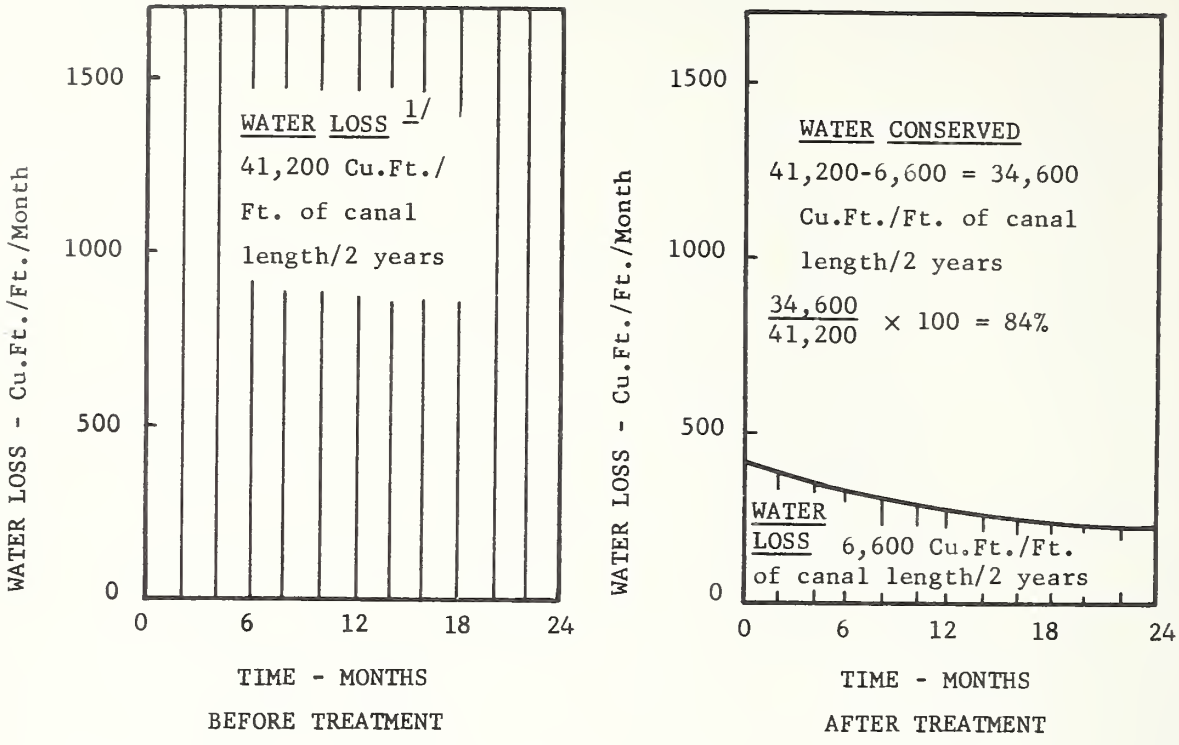


Figure 2.--Seepage rate versus depth, South Canal test, Salt River Project, Phoenix, Arizona. Data from: "An Investigation to Determine the Effectiveness of a Chemical Additive for Reducing Seepage in the South Canal" by Paul E. Ruff, School of Engineering, Arizona State University, April 1961.



1/ Based on an operating water depth of 5.5 ft.

Figure 3.--South Canal water conservation data, Salt River Project, Phoenix, Arizona. Data from: "An Investigation to Determine the Effectiveness of a Chemical Additive for Reducing Seepage in the South Canal" by Paul E. Ruff, School of Engineering, Arizona State University, April 1961.

For the 3-year test period, this cost would be reduced to $0.67 per acre-foot of water saved. The Salt River Project is planning to treat the majority of the South and Consolidated canals in October 1963.

SS-13 was put on the market commercially in 1959, and much has been learned on the capabilities and limitations of the product since that time. Treatment of suitable soils with SS-13 has resulted in seepage reductions to the range between 0.02 to 0.50 cu. ft./sq. ft./day, depending on the initial loss rates. There have naturally been some failures and disappointments as there are with any new concept and product. Generally we learn more from failures than we do from successes, and now the end result can usually be predicted from laboratory and field evaluation of a soil. Soil sealers and seepage evaluations are really entirely new fields of endeavor, and it has not always been easy to state whether a test is a success or a failure, because there are usually several viewpoints on this subject.

The market for chemical sealants in seepage control has been expanding slowly and several hundred installations have now been made. Some small treatments have not been under the immediate control of the manufacturer and the exact total number of installations is not known. The material has been shipped in commercial quantities to Mexico, Nicaragua, Venezuela, Chile, Argentina, Spain, England, Tanganyika, Australia, Pakistan,

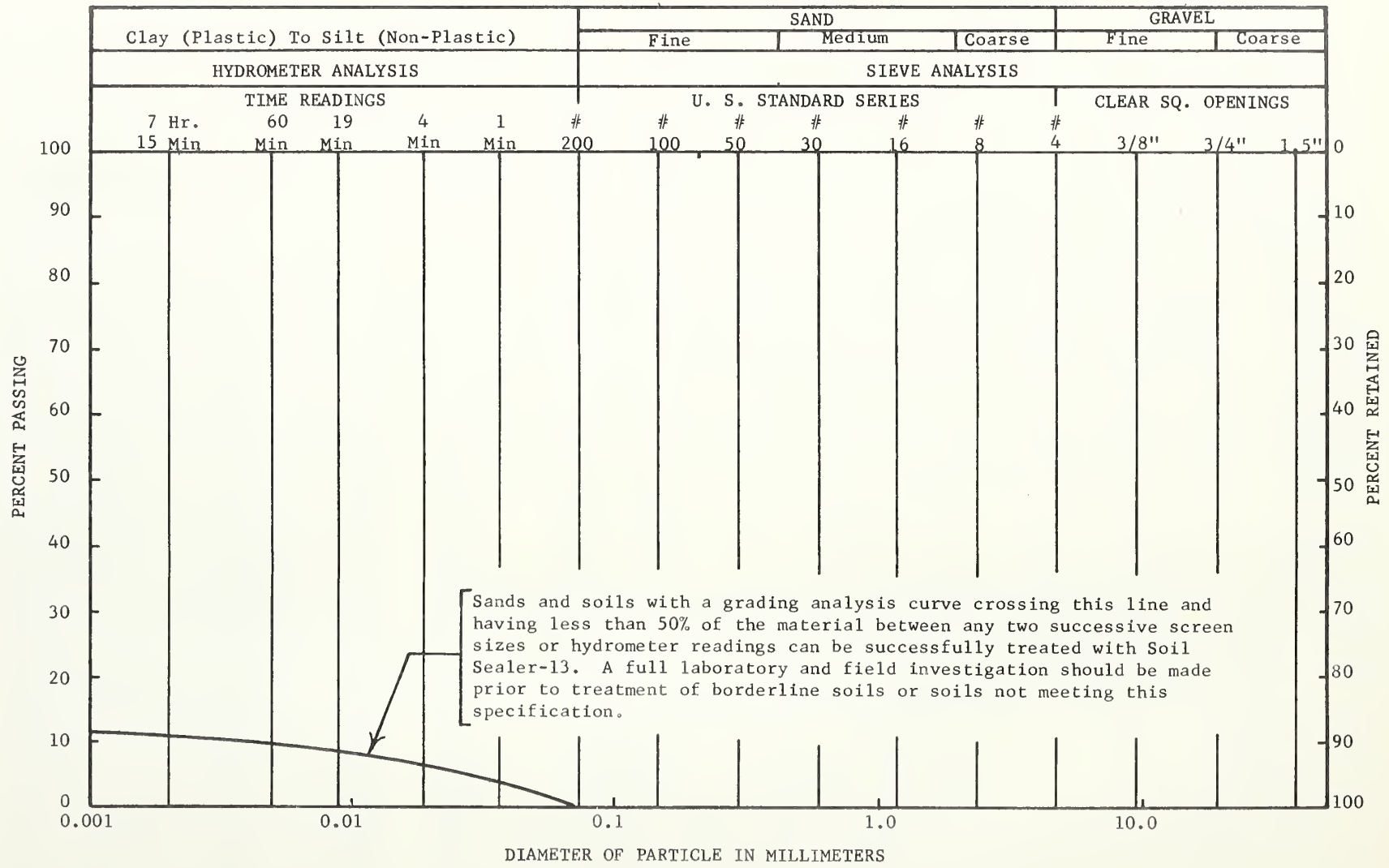
TABLE 1.--Some typical applications of SS-13[1]

Location	Date	Operating water depth	Time of test[2]	Loss rate	Reduction
		Feet		Cu.ft./ Sq./ft./day	Percent
South Canal, Salt River project, Phoenix, Ariz.	Nov. 1958	5.5	PT------- T+1 wk.-- T+1 wk.-- T+2 yr.--	0.94 .28 .16 .12	---------- 70.2 83.0 87.2
Yreka Big Ditch, Gazelle, Calif.	Mar. 1961	4.0	PT------- T+3 mo.--	1.40 .45	---------- 67.2
Kitahara Reservoir, Coachella, Calif.	Oct. 1959	5.0	PT------- PT+1.2 yr. Retreat--	3.50 .63 .17	---------- 82.0 95.2
Lateral 10.2, Boise, Idaho, project:					
Pond 4----------	Oct. 1961	2.0	PT------- T+1 day--	.81 .46	---------- 43.0
Pond 5----------	Oct. 1961	2.0	PT------- T+1 day--	1.40 .42	---------- 70.0
Heartwell Golf Park, Lakewood, Calif:					
Lake 1----------	Sept. 1962	4.0	PT------- T+1 mo.--	1.00 .085	---------- 91.5
Lake 2----------	Sept. 1962	4.0	PT------- T+1 mo.--	.33 .073	---------- 77.9
Scottsdale County Club, Scottsdale, Ariz.	Nov. 1959	---	PT------- T+5 days-	1.10 .27	---------- 75.0
Mission Mine Thickener American Smelting & Refining, Tucson, Ariz.	1961	13+	PT------- T+1 wk.--	3.50 .30	---------- 91.5
Moon Valley Golf Course, Phoenix, Ariz.	Sept. 1959	4.8	PT------- T+25 days	1.10 .30	---------- 72.0
Bureau of Indian Affairs, Lateral Canal, Shiprock, N. Mex:					
Pond 1----------	1961	3.33	PT------- T+4 days-	33.30 .72	---------- 97.9
Pond 2----------	1961	2.50	PT------- T+4 days-	24.40 .15	---------- 99.3
Pond 3----------	1961	2.67	PT------- T+4 days-	22.60 .16	---------- 99.3
Pond 4----------	1961	2.00	PT------- T+4 days-	20.8 .22	---------- 98.7
Gila Gravity Main Canal, Yuma, Ariz. USBR	Oct. 1961	11+	----------	([3])	([3])

[1] Data from SS-13 Sales Co., Phoenix, Ariz.
[2] PT=pretreatment; T+=time between treatment and evaluation.
[3] Efficiency evaluated by observation wells; water surface in all wells adjacent to treated section dropped significantly.

Borneo, Canada, and the Hawaiian Islands in addition to most of our continental states. To my knowledge, the successful treatments constitute about 95 to 97 percent of the total. A few of the treatments and the results are tabulated in table 1.

Some limitations have been set forth as to what soils can and cannot be treated with SS-13. At first, if soil and water samples were sent to our laboratory for prior-to-treatment evaluation, we would make recommendations as to suitability and concentrations necessary. Now a good portion of the laboratory work has been eliminated by the use of particle size determination (ASTM method) and the chart shown in figure.4. It has been a company policy to give a fixed guarantee of results. In general, the guarantee period has not been extended past the first major drying cycle in a treated section. Experience has



DATE: _____ BY: _____ CORE NO. _____

PROJECT: PARTICLE SIZE GUIDE FOR SS-13 TREATMENT

Figure 4.--Chart for particle size determination.

shown that clay content, distribution of particle sizes, degree of compaction, location of the water table, and other factors can change the seepage loss rate if a system is dried up. So far it has been impossible to completely evaluate all of these facets for purposes of a guarantee. Recent data from the Gila Gravity Main Canal at Yuma, treated with SS-13 in October 1961, show that following a 9-day dry up, all evidence of decrease in efficiency due to the dry up disappeared within a period of 90 days. We would like to add, however, that the economics of an SS-13 treatment are such that treatment every year as maintenance can usually be justified even if there is no carryover effect.

The ultimate goals in a seepage control program are the conservation of water which would otherwise be irretrievably lost and prevention of damage by seepage. The present method of evaluating lining materials or seepage control agents on an installation cost per square yard basis is a misleading practice as far as conservation of water is concerned. It is true that the cost per unit of area is necessary in planning a project, but the goal desired is a low cost per unit of water saved. The use of chemical soil sealants can conserve water in many of our existing canal systems for far less cost per acre-foot of water saved than any other method now available. Soil sealants are not a cure-all for all seepage problems, and in some cases cannot compete with conventional lining materials such as concrete and asphalt products. Operational problems such as velocity, sediment loads, weed growth and many others must be considered. The need to remove silt deposits from canals has prompted many questions about the depth of penetration or depth of seal obtained by the use of SS-13. Penetration of any material into fine-grain silts cannot equal the penetration in an open sand for a given exposure time.

Since many types of soil are usually encountered in a short reach of canal, it is impossible to give an average depth of penetration, but the range is generally between 1 inch and 2 feet. Generally speaking, if a canal needs cleaning, it should be done prior to treatment with SS-13. If sediment is deposited after treatment with SS-13, it can generally be removed by the use of conventional mechanical means without too much loss in efficiency. As stated before, the cost of chemical soil sealant treatment is low enough so that it can be treated as a maintenance procedure rather than a capital investment.

The practice of seepage control does not offer the glamour now associated with evaporation suppression from reservoirs and the conversion of sea water, but it does offer the possibility of increasing the delivery from our present gravity water sources by about 50 percent. This figure is based on the well-known fact that about 40 percent of all the water diverted from natural streams for irrigation purposes never reaches the farm, and the conclusion that about 60 to 80 percent of the lost water can be recovered by proper seepage control and water management. To some irrigation districts, the cost of recovering any water by seepage control does not seem justified because they now have the right to all the water they can use and cannot sell the "saved" water. If a district saved 60 percent of the water now lost by seepage at a cost of $2.00 per acre-foot of water saved, and the water was left in the natural stream or reservoir for others to use, the district would have to increase their present rates by about $0.80 per acre-foot. Perhaps this cost is not appealing to an irrigation district, because conservation solely for posterity is seldom attractive to the present generation. In these days of increased competition for water, the public relations value alone might justify the cost. Some districts might justify the cost of seepage control by reduced drainage costs.

Everyone complains about the cost of water, and especially about increases in water rates. Water is still our greatest bargain. In checking my water bill, I found that 7,000 cubic feet of Colorado River water is delivered to my home each month in Metropolitan Los Angeles--filtered, purified, softened and at 100 pounds per square inch pressure 24 hours

a day--for $84.40 per acre-foot, or 6.2 cents per ton. The farmer who pays $4.00 per acre-foot is actually paying 3 cents per ton for his water. Where else can we get such a bargain? It is surely worth the small extra charge for the conservation of this resource to preserve the right to such a treasure.

DISCUSSION

Wednesday afternoon - First Session

Shaw:

I would like to ask Mr. Glenn what is the limiting velocity of water in a canal treated with SS-13 before there is erosion?

Glenn:

We have gone up as high as 4 feet per second without any erosion of material. This is the velocity in the big ditch in northern California, this is the upper limit of our practical experience.

Lucas:

I would like to ask Jack Dybalski a question. What is the minimum temperature you can apply this cationic emulsion?

Dybalski:

I would say about 40^{o} F.

Lauritzen:

I would like to ask Warnick about seepage meter measurements. When you consider the variation in measurements as determined by your seepage meters, is the error in the measurement or is the error in the difference of permeability of the soil from place to place?

Warnick:

In this particular series, I think it was a variation from place to place.

Myers:

I might comment on that question. We ran some ponding tests in which we constructed three or four ditches side by side and divided them up into about 50 ponds 20 to 50 feet long. The visual difference in soils was zero, but the variation in seepage rate from pond to pond was terrific. This indicated to us very forcibly that some of the measurements we had been making with seepage meters were better measurements than they were

thought to be at the time. When we got variations in measurements with seepage meters it was common practice to blame it on the meter; but the ponding tests that we have run, in agreement with what Cal Warnick has said, have shown that seepage varies greatly from point to point in a given ditch, and this is within just a few feet. Some of Herman Bouwer's work has shown this too.

Bouwer:

Not only the seepage varies, but where we have measured the hydraulic permeability of ponded soil, the seepage gradient would also vary tremendousiy.

Van Bavel:

I have a question for Mr. Dybalski. If I understood you correctly you said it is often desirable that the reaction of the cationic emulsion with the soil takes place over a certain minimum depth. You also said that temperature does affect the reaction of this chemical to some extent if both are done by proper formulation. The question is this. Is it possible to combine the temperatures you find at the time of application and the formulation to the proper combination where you get both the desired reaction and the extent with depth as you wish it? Is that a possibility, or are they two different mechanisms?

Dybalski:

They are two different mechanisms. Actually, it is very difficult to get penetration into soils unless they are very permeable because of the characteristics the cationic materials have when they deposit on contact almost instantly.

Van Bavel:

But you did indicate that sometimes it is desirable that it should all be confined to a very superficial layer where reasonable mechanical strength is maintained.

Dybalski:

That is right. But it is really two different mechanisms that we are talking about.

Van Bavel:

Could you elaborate on the nature of these mechanisms?

Dybalski:

First of all, it is difficult to get penetration of the soils because of the inherent nature of the cationic materials. As soon as it contacts a soil particle, the asphalt deposits out. As far as its relationship to deposition in cold water, you get the same thing as a finished product, except you have to alter the formulation so that it will work better in cold water because temperature affects the migratory characteristics of the cationic surfactant, and this just doesn't migrate and adhere as rapidly or as completely as it would in warmer water, but the end-result is still the same. In a silty soil, you still get a surface film regardless of how you do it.

Van Bavel:

In cold water you get no more penetration than you do if the temperature is high?

Dybalski:

No. The only thing that will give you additional penetration is the permeability of the soil itself.

Davis:

With regard to the possible wide use of cationic emulsions in water resources in general, what is the status of your product with the FDA with regard to the possible development of carcinogens?

Dybalski:

The cationic materials themselves are not carcinogenic. Some carcinogenic characteristics have been attributed to the asphalt from time to time, but I think a lot of asphalt has been used in a lot of linings with chemical water. I don't think they have any evidence to substantiate this as far as the carcinogenic characteristics of asphalt itself. Tar is another thing, but with asphalt, it's relatively inert.

Myers:

I might make a very brief comment on that. Aslphalt is listed as a nontoxic material, and the question you asked is now under rather extensive investigation. Within 6 months to a year, hopefully, we will have that answer.

Kirdar:

Is it possible to determine if the characteristic of the finished material is cationic or anionic?

Dybalski:

Yes, by a particle charge test. You simply immerse two electrodes in emulsion and impart a charge across the electrodes and if it is cationic, it will migrate to the cathode; and if it is anionic, it will migrate to the annode; if it is nonionic, it will not migrate at all.

METHODS FOR APPLYING WATERBORNE PETROLEUM SOIL SEALANTS[1]

V. A. Rundle, C. E. Watson, and P. L. Chapdelaine

SUMMARY

The Chevron Soil Sealant Process was developed in close cooperation with the Bureau of Reclamation of the U.S. Department of the Interior. Its purpose is to provide an effective low-cost method for reducing water losses from irrigation systems.

In order to use the process most efficiently, it has been found necessary to develop a number of application methods to suit the wide variety of seepage problems which have been encountered.

[1]Contribution from the California Research Corp., Richmond, Calif.

The Chevron Soil Sealant Process and its various application methods are discussed in detail in this report. All of the methods described have been used in actual field treatments.

THE CHEVRON SOIL SEALANT PROCESS[2]

This process involves the dispersion of a petroleum-based emulsion in the water of the waterway or reservoir to be treated. The diluted emulsion enters the soil wherever seepage is occurring. After contacting the soil to a depth designed in the engineering of the treatment, the emulsion breaks and forms a membrane of very low water permeability. This membrane extends throughout the entire soil area forming the wetted perimeter of the waterway.

The process chosen for a specific treatment is based on an engineering study of the soil to be sealed and on the operating characteristics of the waterway or reservoir. An emulsion formulation is selected which will generate the membrane at the desired depth. This sealant is then produced in the necessary quantity, transported to the treatment site, and injected into the water. All phases of the work are closely controlled.

THE SEALING MECHANISM

The sealant disperses rapidly and uniformly when added to flowing water. As the diluted emulsion reaches a permeable surface, it penetrates into the soil.

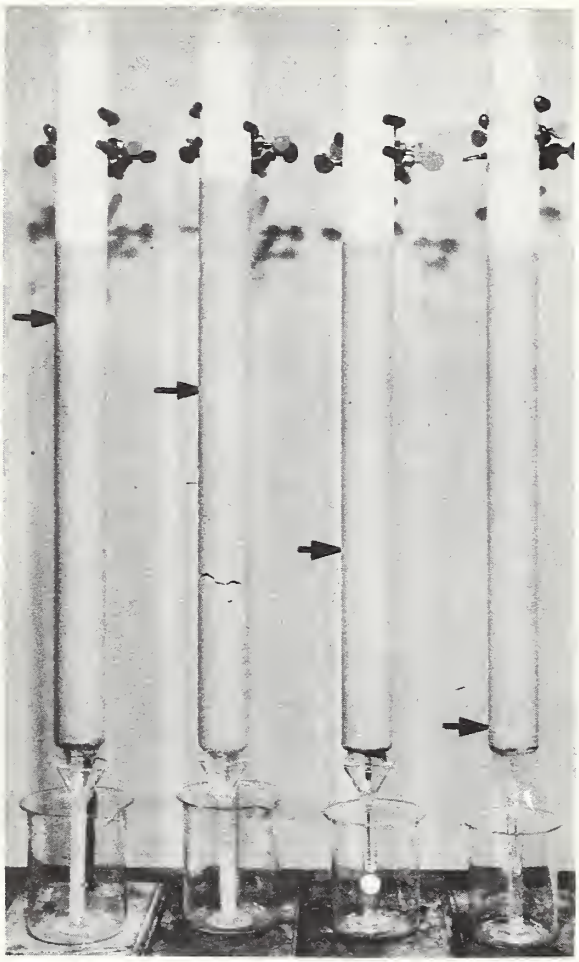
Seals which formed in laboratory soil tests are shown in figure 1. They are about one-sixteenth inch thick and have very low water permeability.

The visual appearance of the thin membrane is accompanied by a marked reduction in seepage. The efficiency of the seal continues to increase. The seepage history of a laboratory test is shown in figure 2. This seal formed 10 inches below the soil surface and effected a seepage reduction of nearly 99 percent after 48 hours. The seepage rate remained normal for the initial part of the test while the emulsion was penetrating to the depth where the seal was formed. In field treatments, the appearance of the soil surface does not change after treatment except in low seepage rate areas where surface deposits often appear. Most of the membrane (fig. 3) in one treatment formed at a uniform depth of about 8 inches, giving a seepage reduction of over 90 percent.

Laboratory studies of the variables involved in the process show that the major parameters are the pretreatment seepage rate of the soil which is usually related to the gradation of the soil particles, and the characteristics of the petroleum-based emulsion used in the treatment. Minor parameters are water temperature, water hardness and pH, chemical composition of the soil, hydrostatic head, and concentration of the sealant. Figure 4 shows the range of natural soil gradations which have been treated successfully. Soils much beyond this range can be treated to reduce seepage but the economics are not as attractive. Those of smaller particle size, although sometimes having seepage problems requiring control measures, do not have seepage rates which are considered to be high enough to create major water losses. Those of larger particle size have such high seepage rates that the amount of sealant required approaches the cost of lining

[2] The Chevron Soil Sealant Process is covered by United States and foreign patent applications assigned to California Research Corp.

FIGURE 1
SEALS FORMED AT DEPTH



the waterways with a physical barrier such as a buried asphalt membrane. However, in our experience to date with nearly 250 soils examined and tested, we have not yet encountered a sample too coarse to seal.

APPLICATION OF THE CHEVRON SOIL SEALANT PROCESS

The background and experience of the Bureau of Reclamation was used for the initial field trials of the process. It was soon found, however, that variants of the standard methods and new methods of application were required to develop the full efficiency and economic potential of the process for treating the water seepage problems that were encountered. Bureau of Reclamation engineers have participated in several Chevron Soil Sealant treatments and have developed a good working knowledge of the application methods that are being used. These methods are discussed in the following section.

METHODS FOR APPLYING SEALANT TO WATERWAYS

A variety of methods have been developed for applying the soil sealant to open waterways. The methods selected for conducting a treatment will depend on the nature of the seepage problem, the design and operating characteristics of the waterway, length of waterway to be treated, amount of time the waterway can be removed from service, and economic limitations if imposed. Each treatment is the result of a special study of the actual problem, but the selection of the method to be used can be generalized as follows.

The Flowing Method

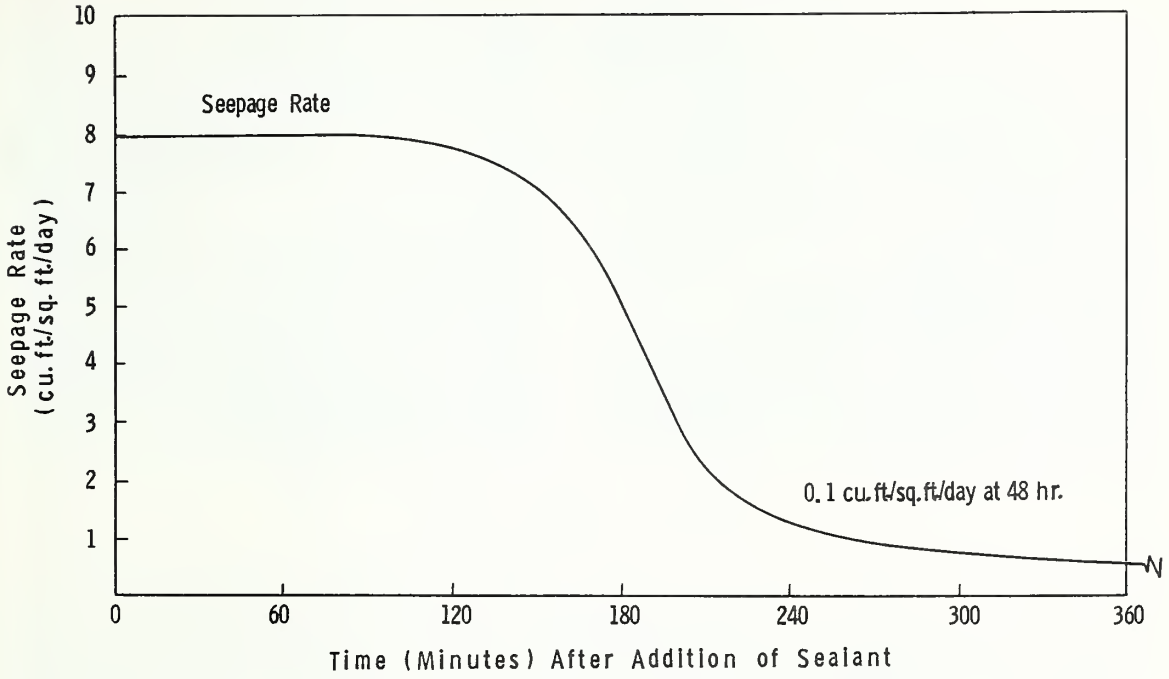
The flowing method is dispersion of the sealant in the water, usually at the headworks and allowing it to flow through the entire waterway.

This method is used when long waterways are involved. If possible, the water level is maintained at or above the design operating level and the velocity reduced. This is done by installing stoplogs in the permanent structures of the waterway.

Application by this method is shown in Figure 5. The typical V-shape of the front end of the sealant slug is starting to develop. The slug gradually becomes longer and more dilute at both ends, but the resultant longer residence time compensates for the lower concentrations.

FIGURE 2

MEMBRANE SEAL FORMATION



The advantages of this method are (1) low labor requirement, (2) minimum out-of-service time for water deliveries from the canal (deliveries are stopped only when the treated solution is at the takeout), and (3) minimum sealant used per unit area treated.

FIGURE 3

EXCAVATED MEMBRANE WEST SIDE LATERAL
EDEN PROJECT, WYOMING

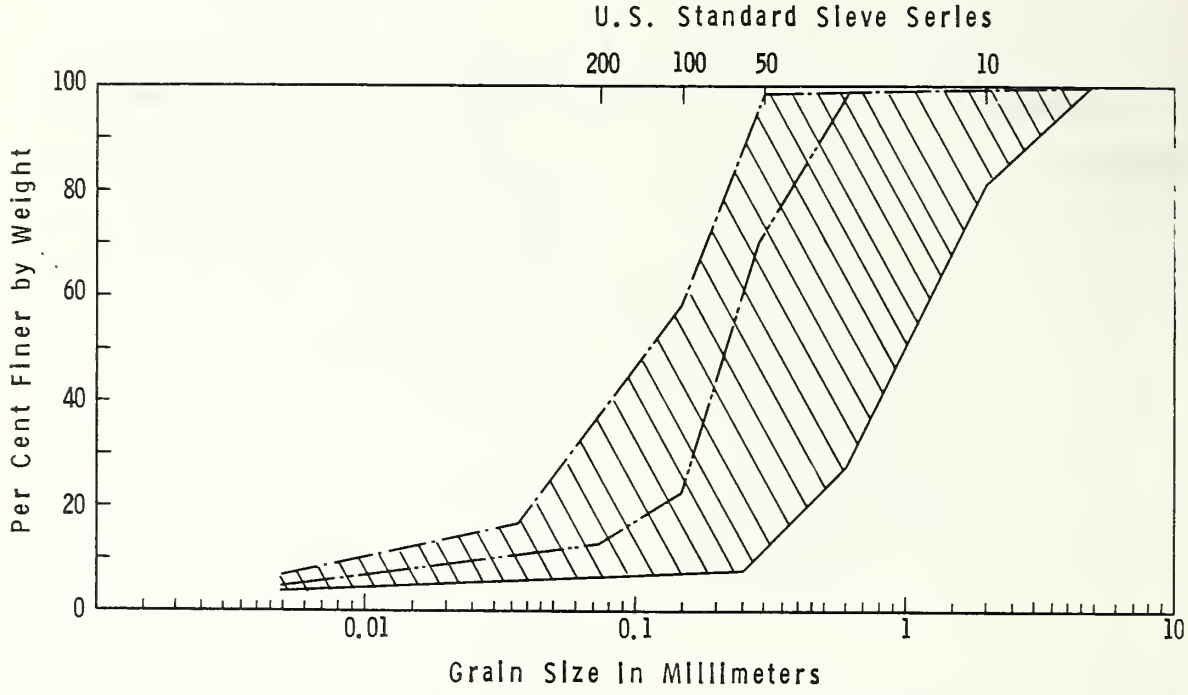
Disadvantages of the method are (1) long flowing distances are required to justify generation of a treating solution slug which will have sufficient contact time at any point along the waterway, and (2) treating efficiency varies with the seepage rate--moderate or high seepage soils are required to obtain high seepage reduction (over 85 percent).

The Ponding Method

The ponding method is dispersion of the sealant, either in the water entering the dammed section of the waterway, or on the surface of the water-filled section. The section is filled to the operating level and the dispersion allowed to stand until absorption is at a very low rate. Additional dispersion is added frequently to keep the water level high. After treatment of the section, the excess dispersion is allowed

FIGURE 4

SOIL GRADATION CURVES SHOWING TYPICAL SOIL TYPES IN WHICH THE SOIL SEALANT GIVES GOOD PERFORMANCE



to flow down the waterway and be absorbed in the bottom areas of the lower reaches.

An application by surface dispersal was made to a 1-mile-long section of the Madera Canal, Madera County, Calif. Temporary earth dams were built at each end of the problem section to allow a full ponded treatment. Pretreatment and posttreatment seepage measurements were made from the normal operating level of 9.5 feet down to 7 feet. At a depth of 7 feet, seepage was reduced from 0.25 to 0.07 cu. ft. per sq. ft. per day, or 72 percent. At the 9.5-foot depth, seepage was reduced from 0.46 to 0.23 cu. ft./sq. ft./day, or 50 percent. The difference is probably due both to the difference in head and to incomplete treatment of the upper side slopes. No attempt was made to obtain a full treatment of the upper side slopes in this case, because seepage was believed to be occurring almost entirely in a permeable stratum below a water depth of 4 feet.

Advantages of the ponding method include (1) high seepage reductions are obtained, (2) problem sections of a waterway can be treated separately using modified sealant compositions and concentrations, and (3) reliable static seepage reduction measurements can be obtained.

Disadvantages involve (1) labor requirements are moderate, since a temporary dam is usually required to generate one boundary of the ponded section; and (2) the out-of-service time for the waterway may amount to several days, making it necessary to treat before or after the irrigation season.

FIGURE 5

CHEVRON SOIL SEALANT APPLICATION, FLOWING METHOD

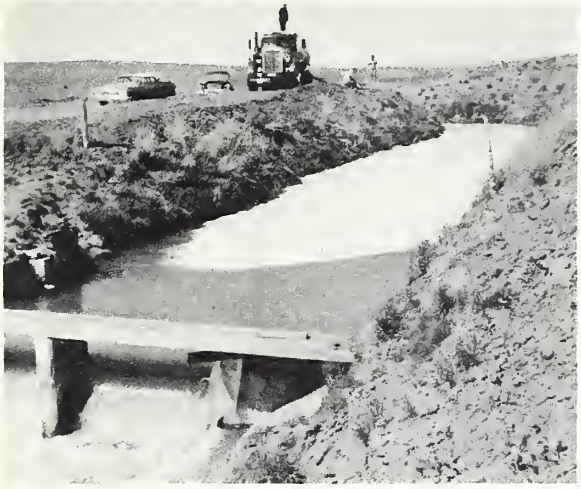


FIGURE 6

SHAW LATERAL, NEWELL, SOUTH DAKOTA, SERIES PONDING METHOD

The Series Ponding Method

The series method is similar to the ponding method but involves construction of several temporary dams at increasing intervals down the waterway. The treating solution is moved successively from one reach to the next in ponding. The location of the next section dam is calculated from the volume of treating solution remaining after the previous ponding is completed.

Figure 6 shows several sections of a lateral filled during the first stage of a series ponding type treatment. Each time the sealant is moved, a shorter distance is treated unless narrowing of the waterway compensates for the decrease in the sealant volume.

Advantages of this method include (1) the inherent high seepage reduction of the ponding method, (2) treatment of a reach for seepage reduction with a minimum amount of sealant, and (3) accurate seepage reduction measurements. The disadvantages of the ponding method are amplified when using this method.

The Bottom Seal Method

This method is a flowing or ponding method where only the bottom of the waterway is treated for seepage reduction. This method is used where it is known that the major part of the seepage is occurring in the bottom. It has the advantage of requiring only a small volume of sealant at a given time. Efficiencies obtained approach those inherent in the basic treatment method involved.

The Combination Method

The combination method includes the bottom seal method followed by the flowing or ponding method. This is used when the soil seepage rate varies markedly with depth. Different petroleum-based emulsions are frequently used for each part of the combination treatment.

A small lateral which had a coarse, clean sand lens across the bottom and sandy loam in the banks was treated. The bottom was treated by ponding to a shallow depth (about 6 inches) with a modified sealant for coarse soil. The excess sealant was drained and the

entire section was then treated to operating level with the deeper penetrating regular Chevron Soil Sealant. Seepage was reduced from 34.4 to 1.47 cu. ft./sq. ft./day, a 95-percent reduction.

The main advantage of this system is that it can be used to treat highly stratified soil. Disadvantages are (1) two types of sealant are required, and (2) more time is required than for other methods.

METHODS FOR APPLYING SEALANT TO RESERVOIRS AND PONDS

The Ponding Method

The same procedure is used as described for sections of waterways.

A 1-acre surface area reservoir was treated by surface dispersal of the sealant to the nearly filled reservoir. Water was running into the reservoir near the point of addition, and the sealant covered the entire surface within 2 hours. Dispersion to the bottom occurred within 24 hours. Seepage was reduced from 0.7 to 0.14 cu. ft./sq. ft./day, an 80-percent reduction.

The Surface Dispersal Method

Where it is not possible to maintain the treatment level by adding water to a reservoir, application to the water surface will help in obtaining adequate treatment of the upper bank areas. The initial high sealant concentration near the water surface allows the banks to absorb more sealant as the water level drops than would be possible if the sealant were completely dispersed from the start.

The Combination Method

The same procedure is used as described for sections of waterways.

Restricted Area Methods

Where seepage losses are known to be occurring through localized soil areas, such areas can be isolated by inserting plastic "fences" or "blankets" into the filled waterway, reservoir, etc. Sealant is then injected into the section, where its dispersion is limited by the plastic barrier. Treatment is conducted as in the ponding method.

Advantages of this method are (1) the marked reduction in sealant required for treating side areas, (2) a minimum contamination of water, and (3) treatment of critical areas. Disadvantages of the method are that (1) it requires special equipment and labor and (2) it is difficult to use in extremely deep ponds or reservoirs.

OTHER USES FOR SOIL SEALANTS

In addition to sealing open unlined waterways by this process, successful results are obtained in treatment of cracked underground concrete irrigation pipes, fractured ceramic sewer pipes, and deteriorated concrete-lined systems. Sealing can also be accomplished in soils not used for water transportation or storage.

Underground Pipes

Leakage from a 24-inch-diameter, buried concrete pipe produced ponds or wet spots and damaged crops at several points in the field above the pipe. A ponding treatment of the pipe reduced seepage by over 90% and eliminated all visible leaks to the soil surface. Sealing is accomplished by the formation of a membrane in the soil in the immediate vicinity of the leak.

Advantages of this method are (1) low labor requirements and (2) high efficiency and low treatment costs. A disadvantage of this method is that yearly treatments may be required to seal new cracks which commonly develop in pipelines each year due to soil movement.

The Flooding Method

This method is used where it is desired to treat level or nearly level areas to conserve water for plant growth or to reduce the subsoil moisture content in order to prevent soil movement. The diluted sealant is allowed to spread over the area to be treated. The depth of the dispersed sealant is usually less than 6 inches. An advantage of the method is that it can be used to treat flat areas not ordinarily wetted. A disadvantage is that dikes are required where the ground is not level enough.

CONCLUSIONS

All of the treating procedures described have been used when appropriate for applying the Chevron Soil Sealant Process. The treatment method is selected to give the optimum results with each specific seepage problem. Wide variations in seepage situations make generalized treatment procedures impractical. Each job is, therefore, considered as a separate engineering study.

SEEPAGE CONTROL USING BUTYL RUBBER LATEX[1]

M. Fefer and J. R. Condon

Harvesting precipitation from rainfall or snowmelt is one of the more promising methods of increasing the ground water supply in the arid lands of the Southwest. Butyl rubber and vinyl plastic membranes, poured concrete and asphalt emulsion sprays are examples of current ways of doing this. Butyl rubber latex can be utilized to control water seepage in any of the following ways:

1. As a lightly pigmented barrier film laid over soil, concrete or other firm surface.
2. As a modifier for concrete.
3. As a modifier for asphalt emulsions.

[1]Contribution from the Enjay Chemical Co., Elizabeth, N.J., and Enjay Laboratories, Linden, N.J.

Two of the above forms, pigmented films and modified asphalt emulsions, can be applied quickly and economically by various spray techniques. The latex-modified concrete could be poured in conventional forms or troweled on in a thin veneer over existing poured concrete.

Before we discuss each of the above applications in more detail, let's look at the properties of butyl latex which make it suitable for this purpose.

PROPERTIES OF BUTYL RUBBER LATEX

There are two butyl rubber latexes commercially available which differ only in total solids and emulsifier content. These latexes are dispersions of butyl rubber in water and possess extremely high mechanical and chemical stability; i.e., the latex cannot be coagulated by the addition of acids, solutions of common alkalis, and metal salts. The latex particularly useful for the purposes of this discussion is described in table 1.

Table 1.--Typical readings from inspections of Enjay Butyl Latex 80-21

Polymer properties:	Enjay Butyl
Total solids.....weight in percent....	53 to 55
Weight per gallon.....pounds....	8.12
Solids.....specific gravity....	0.96
pH.....	5 to 6
Average particle size.....microns....	0.5

FILM PROPERTIES

Films cast from butyl rubber latex are characterized by extremely high elongation, low tensile strength, and surface tack. An important property of butyl rubber is its resistance to embrittlement on prolonged exposure to heat, light and oxygen. The well-known moisture vapor barrier properties of butyl rubber are described below.

Polymer film:	Moisture vapor transmission Gram/100 sq. in./24 hr./4 mils and 95 percent relative humidity and 100° Fahrenheit
Polyethylene	0.23
Butyl latex	.24
Neoprene latex	3.2
Natural rubber latex	5.5

Butyl latex films, even when highly loaded, are extremely good in low temperature extensibility. An example of the elongation of butyl compared to two loaded resin latex films is given in table 2.

Table 2.--Low temperature extensibility of loaded polymer films[1]

Latex	Pigment loading	Maximum elongation at--		
		72° F.	40° F.	20° F.
	Parts per hundred of rubber	Percent	Percent	Percent
Butyl	170	175	1,400	1,225
Do	270	110	1,085	300
Polyvinyl acetate copolymer	185	165	(2)	(2)
Polyacrylate	160	450	(2)	(2)

[1] 1-inch strips evaluated on Instron Tester at 0.5 inch per minute jaw separation rate.
[2] Brittle.

At 72° F., all of the polymer films have approximately the same extensibility (although the polyacrylate film is somewhat better). The resin latex films, however, are so brittle at 40° that extensibility readings cannot be obtained. Butyl films containing as much as 50 percent more pigment give relatively good elongation at temperatures as low as 20°. The reason for the loaded butyl films demonstrating higher elongations at low temperatures than at room temperature is not completely understood. It is believed that several factors come into play at low temperatures which allow the films to elongate further. These factors include: increased crystallinity, increased polymer viscosity, and stronger bond forces between the pigment particles and the polymer.

The films used as illustrations in this table contain much more pigment than would be suggested for optimum resistance to water penetration. However, they serve to illustrate the superior low temperature properties of the butyl latex films.

Even higher extensibility could be expected from butyl films with lower pigment loadings.

COMPOUNDING FOR SEEPAGE CONTROL

It is essential that butyl latex be compounded to yield films with sufficient tensile strength to withstand exposure to wind, sand, rain and light traffic. The proper curatives, fillers, and antioxidants are necessary to maximize performance.

Carbon black imparts resistance to softening by ultraviolet light, but in latex systems does little for tensile strength. Antioxidants should always be used to avoid oxidative degradation. Fillers are used to increase tensile strength and reduce cost. Silica fillers are most useful for improving tensile strength, but must be used at 20 parts per hundred of rubber or less to assure maintaining a water barrier.

The best method of obtaining increased tensile strength is to incorporate vulcanizing agents in the liquid compound. When the films are applied and dried, environmental conditions will catalyze the vulcanization of the butyl rubber. Two different vulcanization systems can be used: lead dioxide/p-quinone dioxime, and a conventional sulfur/accelerator system. The lead dioxide/p-quinone dioxime combination will result in the fastest cures at ambient temperatures while the sulfur/accelerator types give highest tensile strength and elongation.

Table 3.--Results of environmental curing of butyl latex films with lead dioxide/p-quinone dioxime[1]

Days aged	Cast film[2]		
	500 percent modulus	Ultimate tensile strength	Maximum elongation
		Pounds per square inch	Percent
At 72° F.:			
1	145	360	1,300
3	205	360	840
7	225	420	800
30	480	465	530
At 158° F.:			
7	--	455	320
14	--	435	250

[1]

Compound	Dry weight (Percent)
Butyl latex	100.0
Wax	5.0
Amberlac 165*	5.0
Carbon black	5.0
Hydrated calcium silicate	15.0
Lead dioxide	4.0
P-quinone dioxime	2.0

*Resin solution supplied by Rohm & Haas, Philadelphia.

[2] Micro dumbells evaluated on Instron Tester at 10 inches per minute jar separation rate.

Table 3 illustrates the cure progression that occurs with butyl latex films containing lead dioxide/p-quinone dioxime.

This cure system is active and gives a reasonable degree of vulcanization within 24 hours at 72° F. Vulcanization continues for at least 30 days at this temperature. Aging the films at 158° results in more rapid curing with a significant increase in modulus and reduction in elongation.

Table 4 illustrates the effect of aging on the butyl films containing sulfur/accelerators. A temperature of 72° F. is insufficient to activate this system during a 30-day aging period. An increase in the aging temperature to 158°, however, results in a satisfactory cure within one week. Comparison of these data with that in table 3 demonstrates the superior tensile strength and elongation obtained with the sulfur/accelerator system.

Thus, it has been shown that tensile strengths of 300 to 800 p.s.i. with elongations of 300 to 1,000 percent can be obtained with films laid down from properly compounded butyl latex. Care should be taken, however, in choosing the fillers and curatives to yield the film properties desired under the exposure conditions expected. For example, it would seem desirable to use the lead dioxide/p-quinone dioxime cure system where ambient temperatures are not expected to rise above 100° F. The sulfur/accelerator combination would be most useful at 120°-140° where adequate cures could be realized.

FADEOMETER AGING

The effect of ultraviolet light on butyl films containing curatives has been evaluated by aging films in the Fadeometer (table 5). The Fadeometer is operated at 145° F. Black Body temperature and 120° air temperature. The compounds are identical to those listed in tables 3 and 4.

The data show that the compound without curatives is virtually unaffected by the ultraviolet light; but definite vulcanization occurs in the butyl films containing curatives.

Table 4.--Result of environmental curing of butyl latex film with sulfur/accelerators[1]

Days aged	Cast film[2]		
	500 percent modulus	Ultimate tensile strength	Maximum elongation
		Pounds per square inch	Percent
At 72° F.:			
30	45	100	> 2,400
At 158° F.:			
7	300	660	1,000
14	385	810	780

[1] Compound	Dry weight (Percent)	[1] Compound	Dry weight (Percent)
Butyl latex	100.0	Sulfur	1.5
Wax	5.0	Zinc dibutyldithiocarbamate	2.0
Amberlac 165*	5.0	Zinc mercaptobensothiazole	.5
Carbon black	5.0		
Hydrated calcium silicate	15.0		
Zinc oxide	2.0		

*Resin solution supplied by Rohm & Haas, Philadelphia, Pa.

[2] Micro dumbells evaluated on Instron Tester at 10 inches per minute jaw separation rate.

Comparison of these data with that in the previous tables on heat-aged films indicates that the vulcanization is due primarily to the heat generated in the Fadeometer. Similar Fadeometer and Weatherometer exposures of 1,000 hours and exterior exposures of 1 to 2 years have shown virtually no effect of weathering on black butyl latex coatings.

Over relatively rough and porous surfaces such as compacted soil, a minimum of 10 mils (0.010-inch) of dry butyl film would probably be necessary to provide an adequate water seepage barrier. Over smooth surfaces of low porosity such as concrete, 6 mils might well be sufficient. Assuming 10 mils would be necessary, then the raw material cost of a typical black self-curing butyl latex compound would be less than 2-1/2 cents per square foot, or $1,000 an acre. Realistic estimates of application costs can only be obtained when larger scale field tests are made.

Table 5.--Fadeometer aging of butyl latex film

[145° F.--black body]

Curatives	After 22 hours			After 110 hours		
	500 percent modulus	Ultimate tensile strength	Maximum elongation	500 percent modulus	Ultimate tensile strength	Maximum elongation
	Pounds per square inch	Pounds per square inch	Percent	Pounds per square inch	Pounds per square inch	Percent
Check (none)	50	100	2,400	45	80	2,400
Lead dioxide/p-quinone dioxime[1]	--	460	200	--	310	290
Sulfur/accelerator[2]	50	90	> 2,400	220	380	900

[1] Formula in table 3.
[2] Formula in table 4.

CONCRETE MODIFICATION

A second method of utilizing butyl rubber latex in seepage control is as a modifier for concrete to reduce water absorption, increase flexibility, and improve adhesion to certain substrates. The maximum incremental increase in each property is not obtained with any single amount of added latex. Compromises must be made as determined by the dominant properties desired.

Table 6 illustrates the effect of butyl latex content on the water absorption of concrete plugs that were completely immersed in water.

It is apparent that butyl latex does reduce the water absorption, but there is a threshold quantity necessary to obtain a sharp drop. This threshold seems to be exceeded by the addition of 8 percent butyl rubber solids based on concrete weight. This relatively high level of latex would probably be uneconomical in poured concrete but should be satisfactory for application of a thin veneer over existing concrete.

Table 7 illustrates the effect of butyl latex on the hardness and flexibility of concrete. Small amounts of butyl latex greatly improve the flexibility of concrete, but at higher levels the latex reduces the hardness and strength of the concrete. About 4 to 6 percent butyl solids by weight on concrete would appear to give the desired flexibility without excessive softening.

Table 8 illustrates the improvement in adhesion to a porous surface obtained by modifying concrete with butyl latex. About 4 to 6 percent butyl latex solids would appear to give satisfactory adhesion to both paper and cinder block.

Optimum over-all properties would seem to be obtained by adding 4 to 6 percent butyl rubber solids. The improved flexibility and adhesion obtained at this level could well be sufficient to enable the modified concrete to withstand flexing due to substrate changes without cracking and subsequent failure to act as a water barrier.

ASPHALT MODIFICATION

The most economical method of obtaining water-proofing with butyl latex is to utilize the polymer to upgrade the physical properties and weather resistance of asphalt emulsions. Laboratory data given in table 9 show how butyl latex can improve the extensibility of films cast from asphalt emulsions.

Table 6.--Water absorption of latex-concrete blends

Butyl rubber in concrete (Percent of concrete weight)	Water absorption of concrete plugs immersed for--		
	4 hours	72 hours	175 hours
	Percent	Percent	Percent
0.....	9.6	10.8	11.0
2.....	8.4	10.7	12.0
4.....	4.7	9.1	10.4
6.....	6.8	8.2	9.6
8.....	1.9	3.0	4.4
11.....	1.6	2.8	3.7

Table 7.--Flexibility and hardness of latex-concrete blends

Butyl rubber in concrete (Percent of concrete weight)	Flexibility	Hardness
0	Poor	Very hard
2	Fair	Hard
4	Good	Do.
6	Do	Do.
8	Excellent	Slightly soft
11	Excellent	Slightly soft

Table 8.--Adhesion of latex-concrete blends

Butyl rubber solids in concrete	Adhesion to--	
	Paper	Cinder block
0	Poor	Good
2	Fair	Do.
4	Good	Do.
6	Excellent	Do.
8	Do.	Fair.
11	Do.	Do.

Table 9.--Physical properties of asphalt emulsion-butyl latex blends

Asphalt/butyl solids ratio	Ductility	Tensile strength	Maximum elongation
	Centimeters	Pounds per square inch	Percent
100/0	1.8	6.5	90
95/5	4.2	7.8	195
95/5[1]	5.3	6.4	240
80/20	20.8	20.7	> 1,400

[1] Surfactant added.

Table 10.--Water resistance of asphalt emulsion-butyl latex blends

Asphalt/butyl solids ratio	Immersed in water for 3 weeks		
	Water color[1]	Blistering	Wet rub
100/0.....	89.5	Slight	Good
95/5.....	81.7	Considerable	Poor
95/5[2].....	94.5	None	Excellent
80/20.....	92.0	Some	Good

[1] Colorimeter in percent transmission.
[2] Surfactant added.

Five percent butyl rubber increases the ductility, tensile strength, and elongation of asphalt films by greater than 100%. The use of higher amounts of latex results in an even greater increase in properties. When the butyl latex-modified asphalt films are immersed in water, however, it appears that the simple blending technique used results in lower water resistance (see table 10).

This seems to be due to a slight incompatibility caused by the emulsifiers used in the respective dispersions. By adding a suitable nonionic wetting agent, more complete compatibility is obtained with the result that the butyl-modified asphalt demonstrates a clear advantage in water resistance.

The usefulness of butyl latex as a modifier for asphalt emulsion has been demonstrated further by the use of exterior soil flats. These flats, composed of both sandy and clay soil types, were exposed in a horizontal position in New Jersey for a total of 4 years.

The performance of the butyl-modified asphalt, as seen in table 11, is clearly superior to the unmodified emulsion. The butyl latex-modified asphalt applied over sandy soil is practically unaffected after 4 years' exterior exposure. The unmodified asphalt is in relatively good condition, but shows a sufficient number of large cracks that it no longer is an effective water barrier. Although the butyl latex improves the durability of the asphalt emulsion films over clay soil, even these are considered as failing after 2 years.

The application levels used in the above tests amount to approximately 1,500 gallons/acre (~0.050-inch film thickness). At 10 cents per gallon for the asphalt emulsion and $1.60 per gallon for the butyl latex, the total treatment cost would be $215 per acre. Assuming the exposure results obtained with the soil flats are indicative of field experience, then the treatment costs would approximate $50 per acre per year.

The excellent condition of the butyl-modified asphalt emulsion coatings would indicate that relatively low application levels would be necessary for rejuvenation. It would not be unrealistic to expect that only one-fourth the amount used in the original application would be sufficient to rejuvenate the coating for another 3 or 4 years. This could bring the long-range waterproofing treatment costs to $30 per acre per year.

Exterior exposure results by others[2] have indicated the need for pretreatment of the soil to prevent growth of weeds which would penetrate the asphalt. The cost of soil sterilants was estimated at 2 cents per square yard, or $100 per acre. Assuming application would be necessary only once during the lifetime of the artificial watershed, this would yield a total treatment cost of $40 per acre per year for a 10-year period.

[2] Myers, L. E. Waterproofing soil to collect precipitation. Soil and Water Conserv. Jour. 16 (6):281-282. 1961.

Table 11.--Performance of coated soil flats exposed in New Jersey

Coating	Soil	Residual coverage after--		
		2 years	3 years	4 years
		Percent	Percent	Percent
Anionic asphalt emulsion + 3 percent butyl latex.....	Sand	100	100	100
.....Do.....	Clay	95	90	40
Anionic asphalt emulsion.....	Sand	95	95	95
.....Do.....	Clay	85	80	0

SUMMARY

To summarize, then, we have shown that butyl latex can be used to control water seepage in three ways: (1) as a lightly-pigmented barrier film, (2) as a modifier for concrete, and (3) as a modifier for asphalt emulsions. All these approaches take full advantage of butyl rubber's resistance to water penetration, high elongation, and good weathering properties.

EXPERIENCE WITH SEEPAGE CONTROL IN THE PACIFIC NORTHWEST[1]

Francis C. Hart[2]

Seepage as referred to in this discussion is defined as that portion of the diverted water supply which does not reach the place of intended use nor contribute to the production of crops. Only in unusual circumstances does it result in benefits to the project.

Throughout the irrigated West, land suitable for intensive development exceeds the water supply available for such development. As a result, much time has been devoted by responsible technicians to determining optimum amounts of water for full production of climatically suited crops, so that maximum areas of arable land can be irrigated from the limited water resource available. Opportunities for collection and reuse of irrigation water not consumed in initial application have been sought and included in the plans for irrigation development. Quite frequently the seepage losses from the conveyance systems have not received the same technical study as other phases of proposed projects. Instead these seepage losses have been considered as a sort of toll charge that had to be paid in connection with delivery of the irrigation supplies. Construction costs of enclosed conduits or fully lined delivery canals generally have been considered to be excessive for the amount of increase in water deliveries. Where excessive seepage losses have been found in localized areas of delivery works, some efforts have been made to reduce them by lining.

As compared to the areas of short supply and high-value crops in many western irrigation areas, the amount of seepage loss treated as reasonable was relatively greater in the Pacific Northwest in the early years of irrigation operations. Larger amounts of usable precipitation together with relatively abundant natural runoff contributed to this tendency to consider relatively larger seepage losses as economically justified. However, it became apparent that in numerous areas these larger gross losses were

[1]Contribution from the Bureau of Reclamation, U.S. Department of the Interior.

[2]Drainage and ground water engineer, Boise, Idaho.

contributing to more rapid buildup of ground water levels with accompanying need for drainage construction and a requirement for greater capacity in those works. Design standards for canal and lateral systems were changed so as to reduce carriage losses in new construction and means were sought to reduce losses from previously constructed delivery systems.

Experience with seepage losses on some of the older Bureau of Reclamation projects in the Pacific Northwest will be described together with means employed to reduce the losses which originally had not been considered to be excessive. Location of these projects is shown on the map (fig. 1) of the Bureau of Reclamation's Region 1 embracing all of the Columbia River drainage together with the coastal areas of Washington and Oregon including the Rogue and Umpqua River basins in southern Oregon.

MINIDOKA PROJECT - IDAHO

This project was constructed by the Bureau of Reclamation (then the Reclamation Service) beginning in 1904. The project consists of the Storage Division with five reservoirs of 2,784,600 acre-feet total active capacity, on the Upper Snake and Henry's Fork Rivers in Idaho and western Wyoming; the Gravity Division of 72,000 irrigated acres on the north side of the Snake River near Rupert, Idaho, constructed between 1905 and 1907; the South Side Pumping Division, also constructed in 1905 through 1907 with a canal system of 1,325 c.f.s. capacity including three pumping plants of 30-foot lift serving 48,000 acres near Burley, Idaho; the Milner-Gooding Division, served by a 70-mile canal of 2,700 c.f.s. capacity, begun in 1928 and completed in 1937 which furnishes a full supply to 20,000 acres and supplemental supply for 78,462 acres near Shoshone, Idaho; and the North Side Pumping Division consisting of the 13,650-acre Unit A supplied from Snake River by total lift of 163 feet and a canal 4.4 miles long, and Unit B with about 64,000 acres irrigated by 177 deep wells drawing water from the Snake River basalt aquifer, begun in 1952 and only recently completed. Small laterals serve several farm units under each well. Seepage problems on the Gravity Division will be described.

GRAVITY (NORTH SIDE) DIVISION

A gravity canal with a capacity of 1,700 c.f.s. diverting from Snake River to Minidoka Dam extends westerly for 8 miles to provide irrigation water for the 72,000 acres in the Gravity Division operated by the Minidoka Irrigation District. The area is bounded on the south and southeast by the Snake River, on the north and west by a large sagebrush-covered lava bench some 25 feet higher in elevation on which Unit B of the Pumping Division has recently been substantially completed. The land is nearly level north and south, with an uneven slope toward the west averaging 0.5 to 1.0 foot to the mile for the south half of the project, and a little more for the northern portion.

The first settlers found that the soils, ranging from sandy to loamy in texture, were difficult to seed. Because of exposure to prevailing desert winds, adequate moisture could not be maintained and the dry soil and the seed were quickly blown away. An effort was made to raise the ground water levels so that subirrigation could be practiced in the fashion which had been so successful on the Egin Bench near St. Anthony. This practice involved the application of excessive amounts of water to the land in the spring with canal and lateral losses helping to raise the ground water levels to the root zone. Further irrigations during the growing season replaced the amount consumed. For this reason, on the Gravity Division, farm deliveries in early years ranged from 2.5 acre-feet per acre on the heavier soils to 12.0 acre-feet per acre on the sandy soils. With a heavy clay subsoil along the river, much of the project was rapidly afflicted with high ground water levels requiring drainage. Seepage losses from the canal and lateral system then needed to be reduced.

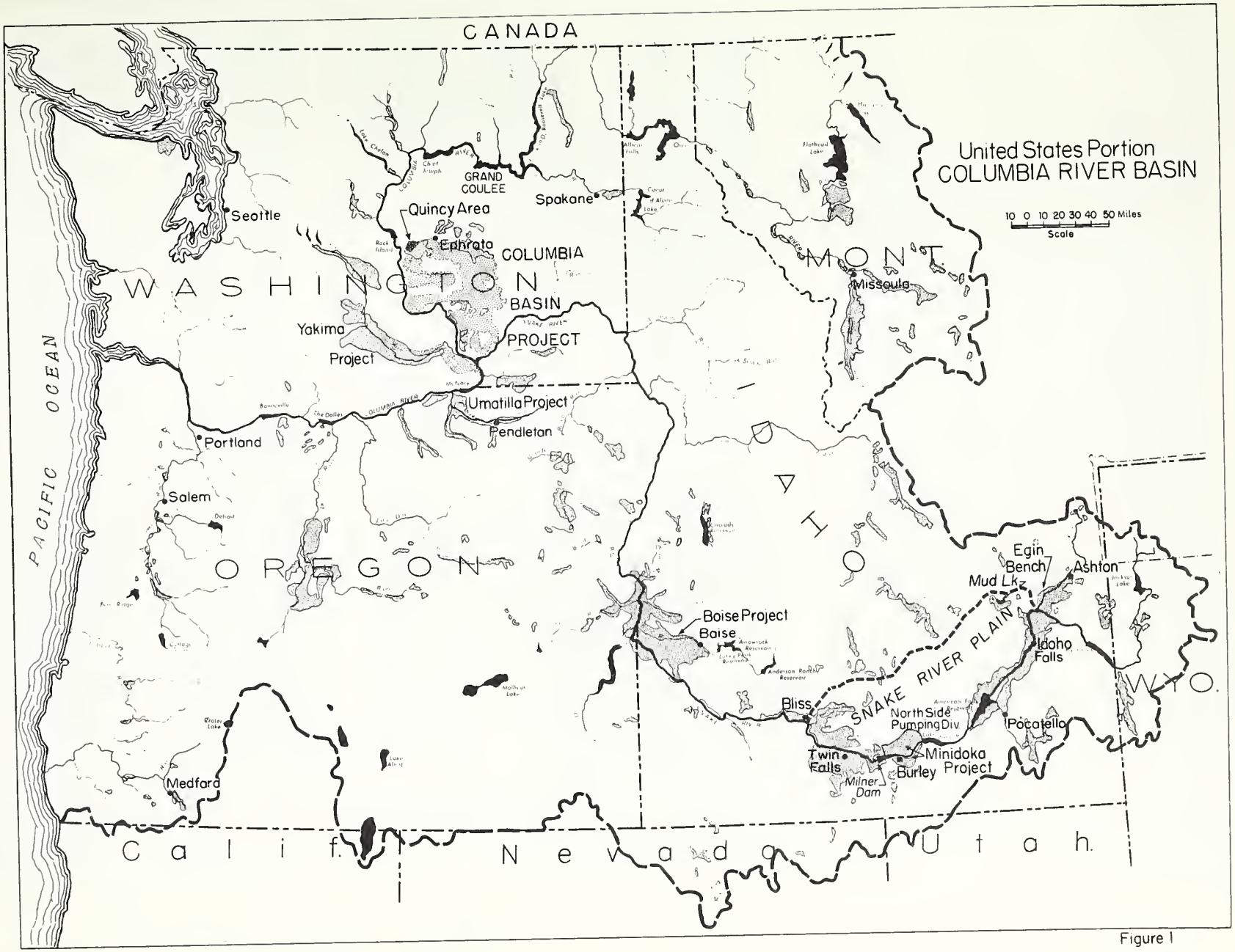

CANADA
United States Portion
COLUMBIA RIVER BASIN
10 0 10 20 30 40 50 Miles
Scale
PACIFIC OCEAN
WASHINGTON
OREGON
Calif.
Nevada
Utah
MONT.
WYO.
IDAHO
COLUMBIA BASIN PROJECT
GRAND COULEE
Seattle
Spokane
Quincy Area
Ephrata
Yakima Project
Umatilla Project
Pendleton
Portland
Salem
Medford
Missoula
Egin Bench
Ashton
Mud Lk.
Boise Project
Boise
Idaho Falls
SNAKE RIVER PLAIN
Bliss
North Side Pumping Div.
Pocatello
Twin Falls
Minidoka
Burley Project
Milner Dam

Figure 1

The flat gradients possible on the canals and laterals required large cross sections with low velocities. They were built largely in fill so that deliveries could be made on both sides. Fill material was generally sandy, so that seepage losses were very high. Total diversion in 1912 was 314,000 acre-feet. Canal losses amounted to 96,000 acre-feet. With allowance of 74,000 acre-feet for consumptive use, lateral loss and waste was 144,000 acre-feet. Of the 314,000 acre-feet, 240,000 acre-feet or 76.4 percent was lost by seepage and waste.

Records kept prior to 1912 were not adequate to permit determination of losses. The losses prior to 1912 were probably larger due to erosional adjustments in the canal cross sections with silt and clay moved as turbid matter into the lower branches of the lateral system leaving clean gravel exposed on the sides and bottom of the channel. The diverted water was clear due to pondage in Lake Walcott.

In 1913, before silting began, measured loss in a 4,000-foot length of the Main Canal amounted to 17.5 c.f.s., equivalent to a rate of 4.28 feet per day per square foot of wetted perimeter. Losses in the entire Main Canal in 1912 and 1913 averaged 110 c.f.s. throughout the irrigation season. In 1914, with silting in progress, this loss was reduced by 14.5 percent to 94 c.f.s. and by 35.5 percent to 71 c.f.s. in 1915. The saving was attributable entirely to the silting.

At the time silting operation was unique for the purpose intended. A deposit of clay estimated to contain 100,000 cubic yards of suitable material was located close enough to the canal to permit mining, processing, and pumping the material into the canal. This deposit, covered with about 2 feet of sand, consisted of a mixture of clay, sand and lime carbonates with a specific gravity of 2.55. The formation had a depth of from 8 to 15 feet. Later a second deposit was located and utilized to supplement the first pit. Here the clay formation ranged from 18 to 45 feet in thickness. Due to the presence of a sticky phase, it was more difficult to handle after mining.

The mechanical analysis of samples from the deposits of canal silting pits was as follows:

Screen size	Percent passing	Screen size	Percent passing
1/4	100	70	98.95
10	99.9	80	98.74
20	99.75	90	98.55
30	99.62	100	98.4
40	99.5	200	96.5
50	99.4	300	92.7
60	99.2		

Material in the deposit was found to be so thoroughly compacted that it was not moved by a 3-inch hydraulic giant nozzle operating at 60-lb. pressure. It was, therefore, found necessary to blast the material from the face of the pit and wash the broken material away from the pit face with continued agitation by the jet until delivered to a storage pond from which the mixture of clay and water was discharged through a delivery structure into the Main Canal. In the 2-year operation, a total of 112,000 cubic yards was mined and pumped into the canal.

By the end of the second year of the silting operation, the silt had been carried through the full length of the Main Canal and to the ends of the lateral system. In fact, evidence indicated that some of the clay had been carried onto the irrigated fields, where its addition to the fine sandy soil proved beneficial in increasing moisture-holding capacity and in better spreading of the irrigation applications by reducing infiltration capacity of the sandy soil surface.

In preparation for the silting operations, bank repairs to the Main Canal were made to improve the results of the silting. Where the canal side slopes had been eroded by higher velocities, by wind-induced wave action, or by curvature in the alignment, the eroded slopes were restored and a blanket of brush held in place by wire mesh was placed to prevent further erosion. A satisfactory amount of silt was thus deposited in the brush blanket to provide an adequate sealing of the side slopes. In other areas, where erosion of the silt appeared likely, the canal section was plowed and harrowed extensively to secure thorough mixing of the canal-deposited silt with the sandy materials of the canal banks and bottom.

The silting operation was accomplished at a cost of $25,000. Before the silting was started, it had been necessary to build a $622,000 drainage system to remove, by way of the drains, a constant flow of 110 c.f.s. of excess ground water resulting from excess irrigation. Assuming that drain costs were proportionate to drain yields, elimination of about 4 c.f.s. of seepage would have justified the total cost of silting.

BOISE PROJECT, IDAHO

This project is located in the valleys of the Boise and Payette Rivers between the mountains and the Snake River. First irrigation was practiced on the Boise River at the townsite of Boise in 1864. By 1900, the irrigated area had grown to 148,000 acres, all irrigated by direct diversion from the Boise River. This early development was accomplished by private companies and cooperative organizations. The construction of the Boise Project by the Reclamation Service was authorized by the Congress in 1905.

As constructed, the project comprises the Arrowrock Division, with 276,000 acres irrigated from the Boise River, and the Payette Division, with 81,000 acres irrigated from the Payette River. Altogether there are 224,000 acres of project land plus 133,000 acres of special and Warren Act lands irrigated from five storage reservoirs with an active capacity of 1,693,000 acre-feet.

There are 557 miles of canals, seven pumping plants, 1,170 miles of laterals, and 395 miles of constructed drains. In many places, especially in the Pioneer Irrigation District and below, it has been found necessary to drill wells and pump ground water to supplement the relief provided by the drainage systems. Where possible, the pumped drainage wells have been located so that their discharge can be delivered to the distribution system as part of the supplemental irrigation supply.

In 1911 and 1912 detailed studies of seepage losses were made cooperatively by the Bureau of Reclamation, U.S. Department of the Interior, and the Office of Experiment Stations, U.S. Department of Agriculture, as a part of a joint investigation by the latter agency and the state engineer of Idaho on the duty of water. A study was made on seven Boise Valley canals under which the irrigated areas were substantially complete and diversity and relative area of various crops grown were considered representative of the then-existing irrigated areas of southern Idaho and adjacent areas of similar climate (table 1).

This study was made to determine the total diversion requirement and its seasonal distribution with no restrictions placed upon the farmer's operations in irrigating his land. A careful census of crops and irrigated area was made. Similar studies were made concurrently on three systems representative of conditions on the upper reaches of the Snake River.

Table 1.--Data on selected canals of upper Snake River, 1911-12

Name of canal	Location on river	Length	Irrigation area	Type of land	Principal soils
		Miles	Acres		
Riverside.....	South side		9,000	Bench	Clay loam.
Farmers Co-op...	North side	28	1,300	--do--	Clay loam to sand.
Farmers Union...	--do--	20	7,000	Bottom	Do.
Settlers	South side		12,000	Bench	Clay loam.
Boise Valley....	North side		2,600	Bottom	Sandy loam.
Pioneer.....	--do--	3	1,265	--do--	Do.
Eureka.....	South side		2,000	--do--	Do.

The results represent the amount of water needed for full crop production under actual project conditions. The studies showed average diversions for the 10 areas as follows:

Month:	Acre-feet per acre	Percentage of total	Month:	Acre-feet per acre	Percentage of total
April					
1-15.....	0.08	1.49	September.....	0.67	12.43
16-30.....	.22	4.08	October		
May.....	.84	15.00	1-15.....	.20	3.71
June.....	1.21	22.43	16-31.....	.05	.03
July.....	1.23	22.83			
August.....	.89	16.50	Totals.....	5.39	98.5

These figures when compared with experimental data on water requirements for the crops being grown indicated that total losses between diversion and application to the land were much larger than had been anticipated.

CANAL AND LATERAL SEEPAGE MEASUREMENTS

To establish, if possible, a sound basis for determining a dependable unit of measure for seepage losses from canals and laterals of different materials, an extensive study under rigid controls was made on constructed systems in southern Idaho. The losses as determined were all reported as "cubic feet lost per 24 hours for each square foot of wetted area in the canal bed." They were reported also as "per cent of loss per mile" since this measurement had been used locally for several years, though the results were frequently misleading. A total of 118 separate seepage measurements were accomplished covering almost every condition likely to be encountered in the area, from farm laterals with capacities of less than 1 c.f.s. to main canals with capacities of almost 3,000 c.f.s. A total length of 287.31 miles of channel were included in the measured sections. A number of interesting conclusions were drawn from the investigations:

1. Great differences were found in the amount lost by canals constructed in different types of soil.

2. Farm laterals carrying 1 c.f.s. or less were subject to abnormal losses when used for conducting water, even for short distances. The value of rotation, where the necessity of carrying small amounts is eliminated, was clearly shown.
3. Certain types of soil have a fairly uniform loss per square foot of canal bed, but the loss in percent per mile was misleading.
4. Other things being equal, if seepage loss is to be reduced, canals should be constructed with as small a wetted perimeter as possible in comparison with their cross section.
5. Velocities high enough to erode and scour the banks increase seepage losses.
6. Other things being equal, there was less seepage loss where natural drainageways were used for conveyance than in constructed channels.
7. Canals in southern Idaho soils under average conditions were found to have seepage loss rates for different materials as follows:

Type of soil in which canal was constructed:	Loss rate throughout wetted perimeter: Cu. ft. per sq. ft. per day
Medium clay loam	0.5 to 1.5
Impervious clay	0.5
Medium soils	1.0
Somewhat pervious soils	1.5 to 2.0
Gravel (depending on porosity)	2.5 to 5.0

8. A comparatively long main canal may lose as much as 30 to 40 percent of diversions even to impervious soils.

On the Boise project the cost of added supplemental supplies from new storage reservoirs increased with time. Also, additional deep drains and supplemental drain wells have been required to control the rising water table. As enlargements of the New York (South Side Main) Canal were made, concrete lining was installed in pervious soil areas as well as in locations where required maintenance of unlined sections was deemed excessive. No uniform criteria have been adopted for determining when lining should be installed in the project canals operated by the Boise project Board of Control. Many short reaches of the canals have been lined, however, on the basis of economic studies which gave cognizance to the many savings attributable to reduction in the seepage losses from the canals.

COLUMBIA BASIN PROJECT

Located in central Washington about 90 miles west of Spokane, 240 miles east of Seattle and from the Columbia River southerly to Pasco at the junction of the Snake and Columbia Rivers, the Columbia Basin project will irrigate 1,029,000 acres when completed as authorized. It is a multiple-purpose development providing irrigation, flood control, power, recreation, fish and wildlife and water quality control benefits.

The project's major features include Franklin D. Roosevelt Reservoir; the Grand Coulee Dam, power and pumping plants on Columbia River; the equalizing reservoir and dams in the Grand Coulee; the Main, West, East-High, and East-Low Canals; O'Sullivan Dam and Potholes Reservoir; and the Potholes East Canal. Many of these features are the largest of their kind in the world. The East-High and East-Low Extension Canals with related works and the Snake River pumping plant for blocks 33 and 34 have been deferred for future construction.

Construction of the project was begun at Grand Coulee Dam in 1933, under the National Industrial Recovery Act, followed by specific Congressional authorization in the Rivers and Harbors Act of August 30, 1935. The Columbia Basin Project Act of March 10, 1943, reauthorized the project, bringing it under the provisions of the Reclamation Project Act of 1939.

Construction of irrigation facilities on the project began in 1946. First delivery of water was for 5,400 acres in irrigation block 1 in 1948 by the Pasco Pumping Plant on Columbia River. In 1950 the Burbank Pumping Plant on Snake River began delivery of water to 1,200 acres in block 2. These units are independent of the primary irrigation system of the project. In 1951, upon completion of the pumping plant at Coulee Dam, six 65,000-hp. pumps began pumping to the equalizing reservoir and in the spring of 1952 the first irrigation water was delivered to the irrigation system then serving about 66,000 acres. Since 1952, extension of the major canals and construction of the lateral system has gone forward on a regular schedule (table 2).

The constructed canal and lateral system is now approaching the point of completion for the irrigable area that can be developed within the cost limitation of 280 million dollars set in the Congressional authorization. There are some 290 miles of canals, 1,600 miles of laterals, 120 pumping plants and 12 relift plants, which made water available to 442,422 acres in 1961.

In general, the canals and laterals have been constructed to the capacity which will be required to serve the total irrigable area thereunder. As designed and constructed, delivery of irrigation water to only part of the acreage requires checking of the ditches to the operating level for full discharge. This results in wetted perimeters considerably larger than would be anticipated. The seepage losses consequently are a larger proportion of the discharge than would ordinarily be the case. In other words, the losses comprise a greater percentage of the supply than would be expected; in fact, they approach the losses which are anticipated for full-capacity operation.

Surface soils of the project are derived principally from wind-laid materials, although some soils in the southern portion of the area have developed on old lake beds and in

Table 2.--Acreage available and actually irrigated, Columbia Basin project, 1948-61

Year	Area for service	Area irrigated	Water diverted[1]	Water delivered
	Acres	Acres	Acre-feet	Acre-feet
1948		230	3,300	750
1949		2,400	26,000	8,700
1950		4,350	30,000	14,000
1951		5,560	45,000	21,000
1952	66,200	27,100	[2] 616,000	122,000
1953	124,700	64,000	[2] 751,000	289,000
1954	180,700	104,000	[2] 1,591,000	475,000
1955	247,700	149,000	1,310,000	650,000
1956	303,000	175,000	1,245,000	762,000
1957	349,700	203,000	1,257,000	825,000
1958	386,400	238,000	1,503,000	1,050,000
1959	411,200	267,000	1,406,000	1,058,000
1960	425,500	294,000	1,639,000	1,219,000
1961	442,400	314,000	1,683,000	1,349,000

[1] Includes dilution water during winter months after 1951 and water for testing and priming new canals and laterals.
[2] Excess diverted from Lake Roosevelt to storage in equalizing reservoir. First water-diverted through West and East-Low Canals, 1952.

smaller part on glacial outwash materials. The entire area is underlain at depths of a few feet to a few hundred feet by the Columbia River basalts. Preproject water table elevations were at considerable depth. It was predicted that deep percolation losses, plus delivery losses and waste would move downward to the basalt and be transmitted laterally to Lower Crab Creek and the Columbia and Snake Rivers without appreciable buildup of ground water levels. Need for deep drains in the irrigable areas was anticipated to be minor for a project of such size.

Experience to date has shown that the original conclusions were too optimistic regarding drainage requirements. Delivery losses from the canals and laterals, surface waste from irrigation, and deep percolation from excessive water applications have resulted in rapid rise of the regional water table. In the lake-bed areas perched water tables have developed. These conditions led to more extensive drainage construction at greater costs. Consequently, need for canal and lateral lining to reduce seepage and the size and cost of drains has increased. Economic studies show that the initial 3.5 cu. ft. per sq. ft. of wetted perimeter design criteria for inclusion of lining are too liberal for the current operating conditions. It has become evident that shorter reaches of canals and laterals must be considered in locating areas of excessive seepage, where lining will yield large benefits in smaller required drains.

Experience shows that even good concrete lining in some of the structureless silts will leak enough water to saturate the silts with sufficient resultant consolidation to cause lining failures due to loss of support for the sub-grade. Deep percolation from farm irrigation above some reaches of lined canal also has caused failures of concrete lining. Compacted earth linings are being used in localities where suitable materials are available.

DISCUSSION

Wednesday afternoon - Second Session

Myers:

I would like to comment on two things that Mr. Hart said. One, he discussed a beneficial use for seepage and reminded us that seepage is not always harmful. Another thing is that if we could do away with gophers, crayfish, bermudagrass, johnsongrass, and cattle, we wouldn't have any trouble in developing a really low-cost canal lining. If somebody can solve those problems, we will have the lining problem solved. I remarked when we first started out that we had anticipated about 50 people, and I said that some of us who have worked on seepage problems for a long time have sometimes been a little bit discouraged. I think those of us who are here are no longer discouraged because the people who are here and the interest that has been shown indicate that something is going to be done about seepage problems.

Warnick:

A question for Vic Rundle: Have you established yet any good means of identifying this emulsion layer? Is there any chemical means of quantitatively identifying how much is in the canal?

Rundle:

We do not have a chemical method. I can give you another way besides the water spray. You can light a match to it and it will often burn, or it will change color. The material will also wet the dry soil and darken it.

Slotta:

To depart from this discussion, I want to ask what work, or what research needs to be done in the ground water and seepage field? We have considered, more or less, the industrial, the municipal, and the state experimental aspects of seepage control; perhaps there should be some discussion on possible research that still remains. A few topics to be considered are the economy and the effect of roughness on percolation within a flowing system. Another one that we haven't discussed here, but I imagine will come up at a future date, is the contamination to wells, through detergents near a municipality or through recharge. Acknowledging actual flow departure from the theoretical Darcian flow--what really exists? What really happens? Perhaps a discussion can be made on this. Then, going back to the first paper of the session--what about the practical depth of well systems in the semiarid and arid West? Does someone desire to discuss the problems of research that could be continued or carried out by academic institutions?

Myers:

If we wanted to discuss things that need research we could keep writing a list for a long time. This session was devoted primarily to agricultural aspects of seepage. If you want to consider seepage as a phenomenon rather than the particular aspect of it that we have in mind here, then the list of problems becomes almost endless. You have just encouraged me very much as we were hoping we hadn't run out of things to discuss at another seepage symposium.

REGISTERED ATTENDANCE

Adrian, Donald Dean	Department of Civil Engineering, Stanford University, Stanford, Calif.
Allen, V. Jay	SCS, 1479 South Wells Avenue, Reno, Nev.
Anderson, Duwayne M.	University of Arizona, Tucson, Ariz.
Baldwin, Ted	Box 134, San Carlos, Ariz.
Ball, Ronald	1113 East Malibu Drive, Tempe, Ariz.
Barnes, S. M.	P.O. Box 877, Corcoran, Calif.
Bendixen, Bill	U.S. Public Health Service, 4676 Columbia Parkway, Cincinnati 26, Ohio
Blackwell, Ben R.	Tipton & Kalmbeck, Inc., 300 Insurance Building, 831-14th Street, Denver, Colo.
Blanchard, Bruce	USBR, 903 North Second Street, Phoenix, Ariz.
Blass, Leon R.	USBR, 903 North Second Street, Phoenix, Ariz.
Bouwer, Herman	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Bower, C. A.	ARS, U.S. Salinity Laboratory, P.O. Box 672, Riverside, Calif.

Breckman, R.	Wellton, Ariz.
Brown, George M.	Brown-Mud Co., Box 2037, Torrance, Calif.
Brown, Kenneth A.	P.O. Box 4425, Phoenix, Ariz.
Brownscombe, R. H.	SCS, 1680 N.W. First Street, Gresham, Oreg.
Brust, K. J.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Busch, Charles D.	Agricultural Experiment Station, University of Arizona, Tucson, Ariz.
Carreker, John R.	ARS, Box 1309, Athens, Ga.
Chiarella, Joseph V.	BIA, P.O. Box 7007, Phoenix, Ariz.
Cluff, C. Brent	Institute of Water Utilization, University of Arizona, Tucson, Ariz.
Condon, John R.	Enjay Laboratories, Box 45, Linden, N.J.
Coons, Lee	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
Cowan, Alex P.	Arizona Gunite Builders, 11023 North 22d Avenue, Phoenix, Ariz.
Coyle, J. J.	SCS, Engineering Division, Washington 25, D.C.
Crittenden, E. D.	Allied Chemical, P.O. Box 61, Hopewell, Va.
Curry, R. Bruce	Department of Agricultural Engineering, Ohio State University, Wooster, Ohio
Davis, John R.	Stanford Research Institute, South Pasadena, Calif.
Decker, Rey S.	SCS, Soil Mechanics Laboratory, 800 J. Street, Lincoln, Nebr.
Dee, W. H.	Box 2342, Page, Ariz.
Dirmeyer, R. D., Jr.	Civil Engineering Department, Colorado State University, Fort Collins, Colo.
Donnan, W. W.	ARS, Southwest Branch, P.O. Box 2326, Riverside, Calif.
Douglas, Ernie	Arizona Farmer Ranchman, 434 West Washington, Phoenix, Ariz.
Douglas, Harry	BIA, Billings, Mont.
Dybalski, Jack N.	Armour Industrial Chemical Co., 8401 West 47th Street, McCook, Ill.
Edmiston, Harlon R.	1921 West Holly, Phoenix, Ariz.
Ehrich, Glen	815 East Jefferson, Phoenix, Ariz.
Erie, L. J.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Evans, John H.	Del E. Webb Corp., P.O. Box 7588, Phoenix, Ariz.
Fasken, Guy B.	1819 East Wood Place, Milwaukee, Wis.
Fefer, M.	Enjay Chemical Co., 1141 East Jersey Street, Elizabeth, N.J.
Foreman, H. J.	SCS, Alexandria, La.
Franzini, Joseph B.	Department of Civil Engineering, Stanford University, Stanford, Calif.
Frasier, G. W.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Freeland, Sten J.	USBR, Region 7, Denver Federal Center, Denver, Colo.
Freeman, Larry	USBR, P.O. Bin 151, Yuma, Ariz.
Frevert, R. K.	Agricultural Experiment Station, University of Arizona, Tucson, Ariz.
Frick, A. L.	Arthur H. Beard Consultant Engineers, 3426 East Indian School Road, Phoenix, Ariz.
Fritschen, L. J.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Fuller, Galen	4425 Marfargoa Drive, Stockton, Calif.
Gain, Elmer W.	SCS, Upper Darby, Pa.
Gardner, J. L.	ARS, Southwest Watershed Research Center, Box 3926, Tucson, Ariz.

Gardnier, E. W.	California Research Corp., 200 Bush Street, San Francisco, Calif.
Garrison, Grove	1017 C VeElle Circle, Tempe, Ariz.
Garvey, Terence M.	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
George, Creed, J.	USBR, 903 North Second Street, Phoenix, Ariz.
Gibson, Alonzo N.	SCS, P.O. Box 1898, Fort Worth, Tex.
Gifford, Falcnor	BIA, 6416 West Glenrosa, Phoenix, Ariz.
Glenn, J. Harlan	Brown Mud Co., P.O. Box 2037, Torrance, Calif.
Glidden, Bill	BIA, Whiteriver, Ariz.
Goldsmith, L. C.	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
Graves, O. Porter	Brown Mud Co., P.O. Box 2037, Torrance, Calif.
Greer, R. M.	Wellton, Ariz.
Griggs, John	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Grover, Ben L.	University of California, Riverside, Calif.
Habegger, H. L.	Globe Linings, Inc., 1901 East Wardlowr, Long Beach, Calif.
Haise, Howard	ARS, P.O. Box 758, Fort Collins, Colo.
Hanson, Eldon G.	Agricultural Engineering Department, New Mexico State University, University Park, N. Mex.
Harris, Karl	ARS, 6036 Federal Building, Phoenix, Ariz.
Hart, F. C.	USBR, Box 937, Boise, Idaho
Hatcher, Otto	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
Hauser, Victor L.	ARS, SW Great Plains Field Station, Bushland, Tex.
Henry, M. J.	8319 Puritan, Douney, Calif.
Hess, D.	Enjay Chemical Co., 615 South Flower Street, Los Angeles, Calif.
Hickok, R. B.	P.O. Box 2326, Riverside, Calif.
Jackson, R. D.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Jenson, C. L.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Jensen, Marvin E.	ARS, P.O. Box 758, Fort Collins, Colo.
Johnson, A. Morgan	4701 North 55th Street, Phoenix, Ariz.
Kam, William	USGS, 4016 Federal Building, Phoenix, Ariz.
Kays, William B.	Globe Linings, Inc., 1901 East Wardlowr, Long Beach, Calif.
Kincaid, David R.	Southwest Watershed Research Center, P.O. Box 3926, Tucson, Ariz.
Kirdar, Ed	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
Koluvek, Paul	SCS, 380 North Eighth Street, El Centro, Calif.
Kruse, Gordon	Foothills Hydraulic Laboratory, Fort Collins, Colo.
Larson, G. H.	Kansas State University, Manhattan, Kans.
Lauritzen, C. W.	ARS, Mechanic Arts 130, Utah State University, Logan, Utah
Lawhon, Lester F.	SCS, P.O. Box 1898, Fort Worth, Tex.
LeCrone, D. C.	829 East Windsor, Phoenix, Ariz.
Lester, W.	Arizona Gunite, 11023 North 22nd Avenue, Phoenix, Ariz.
Lister, Jack	SS-13 Sales Co., P.O. Box 4425, Phoenix, Ariz.
Long, W. E.	SCS, 2020 Milvia Street, Berkeley, Calif.
Long, William F.	SCS, Room 604, Rudge and Guenzel Building, 134 South 12th Street, Lincoln, Nebr.
Lucas, Clifford V.	5210-58th Street, Sacramento, Calif.
Lunin, Jesse	ARS, Box 2160, Norfolk, Va.
Lynch, T. W.	855 North Orlando, Los Angeles, Calif.
Martin, Lewis L.	BIA, Whiteriver, Ariz.
McCorkle, M. R.	Armour Industrial Chemical Co., 8401 West 47th Street, McCook, Ill.

McDaniels, Louis L.	Texas Water Commission, P.O. Box 2311, Capitol Station, Austin, Tex.
McGarrity, John G.	2420 Arden Way, Sacramento, Calif.
McMillen, C. M.	Phoenix Gazette, 120 East Van Buren, Phoenix, Ariz.
Meidinger, Dan	USBR, 903 North Second Street, Phoenix, Ariz.
Millett, Donald F.	556 North Miller, Mesa, Ariz.
Moore, Dave	79 Post, 7th Floor, San Francisco, Calif.
Muckel, Dean C.	ARS, P.O. Box 8014, University Station, Reno, Nev.
Myers, Lloyd E.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Myers, Victor I.	ARS, P.O. Box 267, Weslaco, Tex.
Nakayama, F. S.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Navin, Frank E.	Roosevelt Water Conservation District, Highley, Ariz.
Neese, Paul M.	BIA, P.O. Box 7007, Phoenix, Ariz.
Newlin, Charles W.	Department of Engineering, Arizona State University, Tempe, Ariz.
Nielsen, Donald R.	Irrigation Department, University of California, Davis, Calif.
Nolan, J. F.	American Cyanamid, 515 Walker Bank Building, Salt Lake City, Utah
Ogrosky, H. O.	SCS, South Building, Washington, D.C.
Pampel, L. C.	USBR, 903 North Second Street, Phoenix, Ariz.
Peters, Helen J.	California Department of Water Resources, State of California, Sacramento, Calif.
Powell, Fred	San Francisco, Calif.
Pugh, C. A.	USBR, 903 North Second Street, Phoenix, Ariz.
Ree, William O.	ARS, Box 789, Stillwater, Okla.
Reginato, R. J.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Reisenauer, Andrew E.	General Electric Co., Hanford Laboratory Operations, Richland, Wash.
Resnick, Sol D.	Institute of Water Utilization, University of Arizona, Tucson, Ariz.
Reyskey, R. H.	BIA, P.O. Box 7007, Phoenix, Ariz.
Rice, R. C.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Richards, Alex A.	Salt River Project, P. O. Box 1980, Phoenix, Ariz.
Robinson, A. R.	ARS, P.O. Box 67, Twin Falls, Idaho
Rollins, M. B.	ARS, Box 8014, University Station, Reno, Nev.
Rubin, Jacob	USGS, Water Research Division, Menlo Park, Calif.
Rundle, V. A.	California Research Corp., Richmond Laboratory, 576 Standard Avenue, Richmond, Calif.
Saunders, John D.	BIA, Salt River Agency, Scottsdale, Ariz.
Schadle, Dan	Arizona Game and Fish Department, 1688 West Adams, Phoenix, Ariz.
Schiffler, W. H.	California Chemical Co., 200 Bush Street, San Francisco, Calif.
Schrunk, John F.	5423 Field Ct., Arvada, Colo.
Schultz, Tom	Garwood Industries, Inc., Findlay, Ohio
Shaw, William B.	California Department of Water Resources, State of California, Sacramento, Calif.
Shipley, H.	Salt River Project, Box 1980, Phoenix, Ariz.
Shockley, Dell	SCS, 209 SW Fifth Street, Portland, Oreg.
Sieckman, Don L.	USBR, 903 North Second Street, Phoenix, Ariz.
Skinner, M. M.	Colorado State University Experiment Station, Fort Collins, Colo.

Slotta, Larry S.	Civil Engineering Department, Oregon State University, Corvallis, Oreg.
Smith, D. D.	ARS, Plant Industry Station, Beltsville, Md.
South, G. P.	USBR, P.O. Box 360, Salt Lake City, Utah
Sparks, W. J.	Enjay Laboratories, Box 51, Linden, N.J.
Spencer, William F.	ARS, P.O. Box 2326, Riverside, Calif.
Steegman, R. M.	Garwood Industries, Inc., Findlay, Ohio
Stolzy, L. H.	University of California, Riverside, Calif.
Sutton, John G.	6300 Crathie Lane, Washington 16, D.C.
Sweet, Charles L.	P.O. Box 716, Boulder City, Nev.
Thornton, John	ARS, P.O. Box 208, Columbia, Mo.
Tovey, Terrel R.	BIA, Land Operations, Fort Hall Agency, Fort Hall, Idaho
Turner, J. J.	SCS, 6015 Federal Building, Phoenix, Ariz.
van Bavel, C. H. M.	ARS, U.S. Water Conservation Laboratory, Tempe, Ariz.
Vandertulip, John J.	Texas Water Commission, P.O. Box 2311, Capitol Station, Austin, Tex.
Walker, John	BIA, P.O. Box 7007, Phoenix, Ariz.
Waltersdorf, D. B.	1231 Burlington, Billings, Mont.
Warnick, C. C.	Engineering Experiment Station, University of Idaho, Moscow, Idaho
Warren, C. C.	2323 Harvester Drive, Portland, Oreg.
Welker, Charles H.	Harza Engineering Co., 400 West Madison Street, Chicago, Ill.
Wendt, Stanley	Montague Water Conservation District, Montague, Calif.
Wilder, Carl R.	Portland Cement Association, 816 West Fifth Street, Los Angeles, Calif.
Willhite, Ronald	USBR, 903 North Second Street, Phoenix, Ariz.
Williams, W. L.	60 West 49th Street, New York, N.Y.
Willson, Richard J.	USBR, Building 53, Denver Federal Center, Denver, Colo.
Wilson, L. G.	Institute of Water Utilization, University of Arizona, Tucson, Ariz.
Wilson, Ted	Salt River Project, P.O. Box 1980, Phoenix, Ariz.
Yost, Coyd, Jr.	ARS, Box 400, Chickasha, Okla.

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